PDHonline Course S212 (6 PDH)

Antiquated Structural Systems – Part 2

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2013

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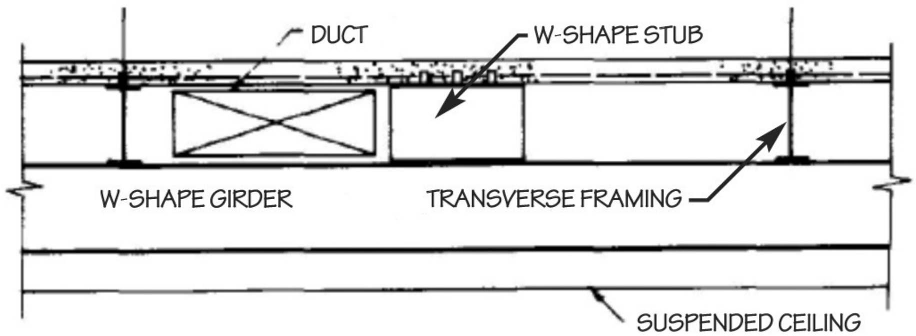
Structural Steel Composite Stub-Girder Construction

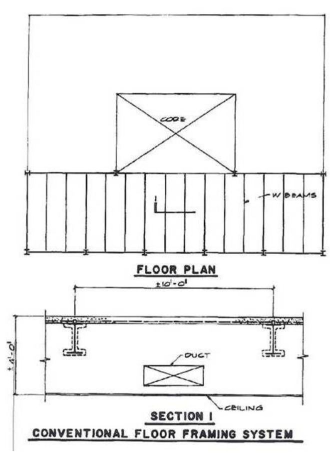
Most of the antiquated systems discussed so far have been out of popular use for a considerable number of years, with some dating back to the first part of the last century. However, the subject of this construction method deals with a system that was still in use less than 20 years ago. (Unless noted otherwise, all images of the Stub-Girder System are reprinted with the permission of AISC)

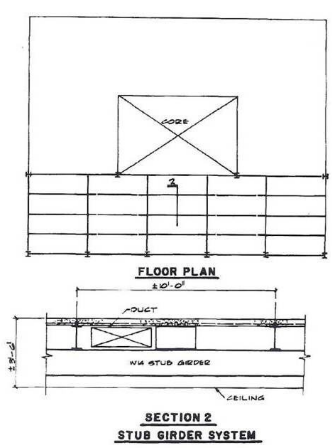


TYPICAL STUB GIRDER ELEVATION

A **Stub-Girder** is a composite system constructed with a continuous structural steel beam and a reinforced concrete slab separated by a series of short, typically wide flange sections, called stubs. The stubs are welded to the top of the continuous beam and attached to the concrete slab by shear connectors. The spaces between the ends of the stubs are used for the installation of mechanical ducts and other utility systems and for placement of the transverse floor beams that span between the stub-girders.

Ideally, the depth of the stubs and the floor beams are identical to allow for the transverse framing to support the concrete slab deck, which spans parallel to the stub-girder, and to facilitate composite action between the floor beam and slab.





Stub-girder construction was first used in 1971 at the 34-story One Allen Center office building in Houston, Texas. The system was developed by Joseph P. Colaco of Ellisor Engineers, Inc., to facilitate the integration of mechanical ducts into the steel floor framing of repetitive, multistory high-rise construction. This system went on to be used in a large number of high-rise buildings in North America up through the 1980's.

However, the system was eventually abandoned because of the increased labor cost associated with both fabrication and the need for shoring until the field-cast concrete slab attained sufficient strength.

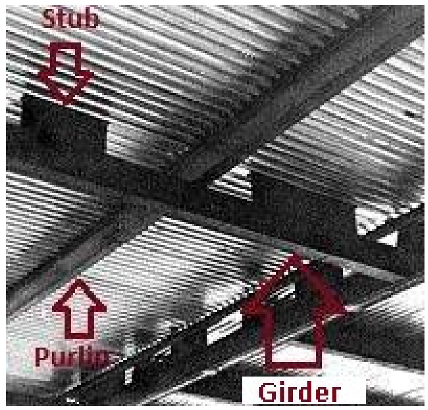
One Allen Center

Source: Vern Gary

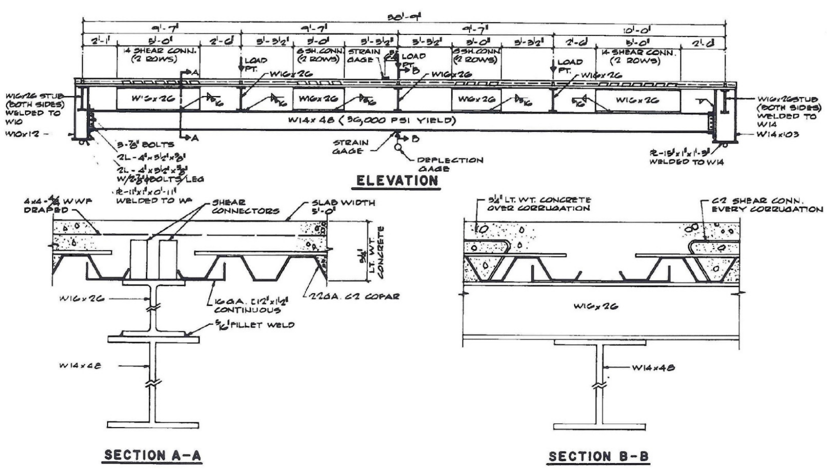
One Allen Center During Construction

Advantages of the stub-girder system that led to its use during the time period in which it was popular included:

1. Reduction in steel tonnage by as much as 25% over conventional composite floor framing due to:
    a. Improved structural efficiency as a result of the greater depth of the stub-girder compared to a conventional system; and…
    b. Improved structural efficiency due to the ability of the transverse floor framing members to act as continuous beams through the openings between the stubs.
2. Reduction of the overall depth of the structural floor framing system by as much as 6 to10 inches over a conventionally framed composite floor system, which allowed for a reduced floor-to-floor height and overall height of the building and associated cladding.

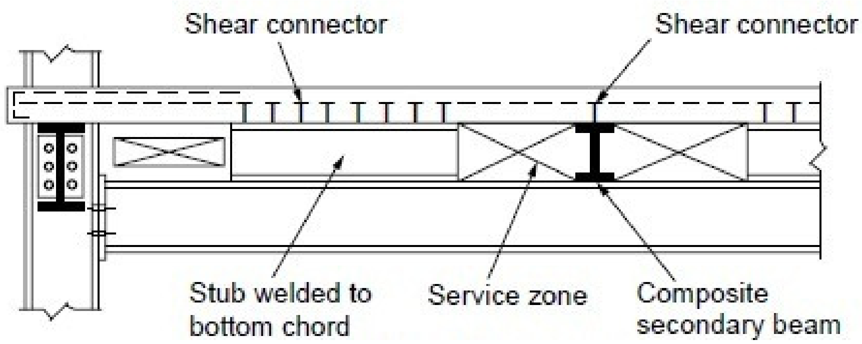


Prior to the use of the stub-girder system, a load test was performed at Granco Steel Products Company in St. Louis. The test specimen included a W14x48 continuous bottom beam, W16x26 stubs and floor beams, and a 5-feet-wide, 3¼-inch-deep, lightweight concrete slab over a 2-inch metal deck flange, which was attached to the stubs via shear connectors.



The test specimen was loaded beyond the calculated design load with the initial failure occurring at the exterior end of the outermost stub at one end of the stub-girder. The method of failure included web crippling and delamination of the web from the flange.

Application of additional load resulted in crushing of the slab at the inside edge of the same stub. However, separation between the bottom of the slab and the top of the stubs did not occur, which indicated that composite behavior was maintained up to the point of localized crushing of the concrete slab. Web stiffeners added to the failed stub allowed the system to achieve a final failure load that was 2.2 times greater than the calculated design load.



The methods of design used to determine the capacity of the section included a non-prismatic beam analysis, a Vierendeel girder/truss analysis and a finite element analysis.

For the Vierendeel analysis the stubs and transverse floor beams act as verticals and the concrete slab and continuous beam act as the chords (see Figure 3 for a comparison of the typical Vierendeel truss and stub-girder components). All three of these methods of analysis provided a close representation of the actual behavior of the stub-girder.

However, the Vierendeel and finite element methods more closely identified the secondary moment effects on each side of the openings. The Vierendeel method of analysis also provided a more accurate representation of the actual steel stress, while the finite element method provided a more accurate representation of the stress in the concrete slab, including the high stresses that resulted in the crushing of the concrete at the inside edge of the first exterior stub as observed in the test specimen.

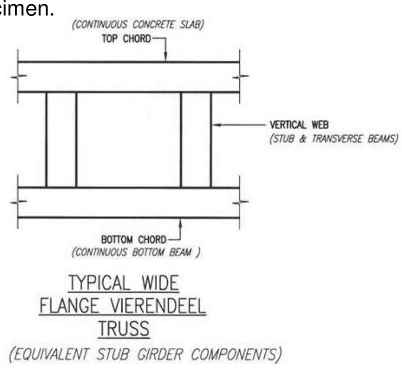
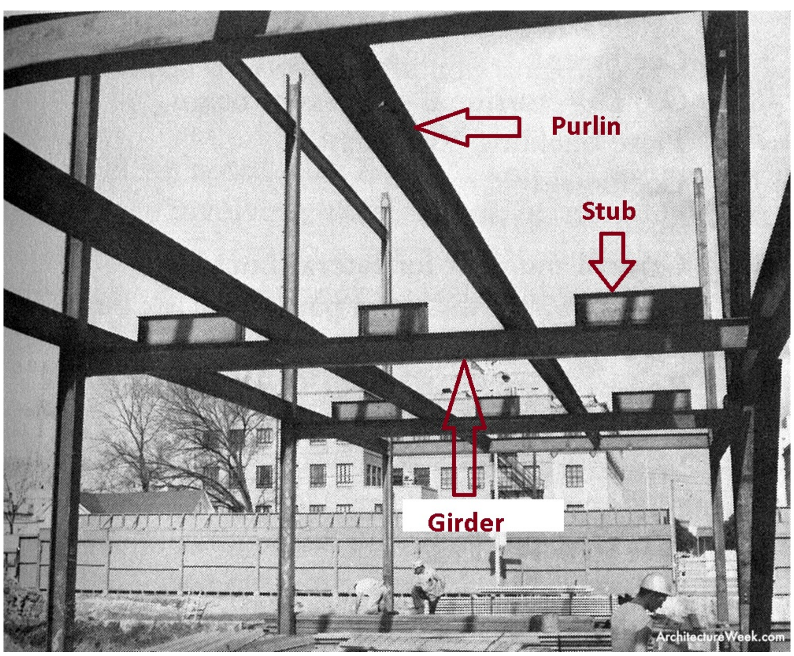


Figure 3

Additional tests of stub-girders were performed in the late 1970's in Canada. The primary purpose of these tests was to determine the effects of changes in the spacing and depth of the stubs and to establish the failure modes of a stub-girder. The results confirmed that the behavior of a stub-girder was similar to a Vierendeel girder/truss. Additional conclusions of the tests included:

1. The stiffness of the girder increases as the length of the open panel between the stubs decreases.
2. Shear distortions at the open panels (as a result of the Vierendeel action) were an important parameter in determining the elastic deflection of the stub-girder, but did not influence the rotation of the solid end sections of the overall girder.
3. Tensile cracking of the concrete slab at the ends of the open panels occurred at relatively low loads, but did not have a significant impact on the elastic stiffness of the girder.
4. Further extensive cracking of the concrete slab at the ends of the open panels occurred in the inelastic range of the girder. It was further determined that the ultimate strength and ductility of the girder could be improved through the use of internal reinforcement within the slab that was placed to resist the observed cracking.
5. The precision of the Vierendeel method of analysis was dependent on the accuracy of the distribution of shear forces between the concrete slab and the continuous lower beam across the open panels and the assumptions made relative to the location of the points of contraflexure within the open panels.
6. Failure of the shear connectors resulted as a combination of shearing and prying effects.
7. To prevent premature failure due to web crippling at the stubs, stiffeners should be provided.
8. Five different failure mechanisms were identified - buckling of the stub web, concrete failure in the vicinity of the shear connectors, diagonal tension failure of the concrete slab, shearing off of the headed stud connectors, and combined yielding of the steel beam and crushing of the concrete slab at the ends of the open panels due to the cumulative effects of the primary and secondary (Vierendeel) moments.



Stub Girder Construction

Further research in Canada revealed additional insights into the behavior, design and economical construction of stub-girders. This research indicated that only partial end plate stiffeners, rather than traditional fitted stiffeners, were required to reinforce the stub webs.

Furthermore, web stiffeners were not always required at the interior stubs. In addition, a continuous perimeter weld between the base of the stub and the top of the continuous beam was not required. The tests also confirmed that rolled wide flange shapes were more conducive to stub-girder construction than split T (WT) or rectangular hollow tube (HSS) sections.

Additional conclusions of these later Canadian tests included:

1. Deflection computations using the Vierendeel method of analysis were typically conservative for service loads and unconservative for ultimate loading conditions.

2. The amount of internal slab reinforcement, particularly in the direction transverse to the stub- girder span, was established based on Canadian Standard Association (CSA) criteria available at the time of the tests.

3. The conventional method of calculating the number of shear studs required and the application of standard methods of composite design to the analysis of stub-girders appeared to provide satisfactory results, however, caution was recommended when specifying closely spaced studs, particularly at the end stub.

Additional recommendations and guidelines emerged throughout the 1980's for the stub-girder system. If fact, AISC had plans to develop a design guide for stub-girder construction; however, because deeper wide flange sections became more readily available and guidelines for the design of reinforced and unreinforced web openings became more established (see AISC Steel Design Guide Series 2; Steel and Composite Beams with Web Openings - 1990), it was never published.

In order to document some of the final design guidelines that were established for stub girder construction, the following list of criteria is provided:

1. Economical spans for stub-girders range from 30 to 50 feet, with the ideal span range being 35 to 45 feet.
2. Transverse floor beams should be spaced at 8 to 12 feet on center.
3. The stubs do not necessarily have to be placed symmetrically about the centerline of the stub-girder span.
4. The use of 3 to 5 stubs per span is the most common arrangement.
5. The stub located nearest the end of the stub-girder (and the surrounding, adjacent truss/girder elements) is the most critical member, as it directly controls the behavior of the overall stub-girder. In addition, the end stub may be placed at the very end of the continuous bottom beam, directly adjacent to the support point.
6. The performance of a stub-girder is not particularly sensitive to the length of the stubs as long as the length of the stub is maintained within the following limits;
    a. Exterior stubs should be 5 to 7 feet in length.
    b. Interior stubs should be 3 to 5 feet in length.
7. However, increasing the length of the open panel between the stubs will reduce the stiffness of the stub-girder.
8. Stub-girders must be constructed as shored composite construction in order to take full advantage of the concrete slab top chord. In addition, because of the additional dead load imposed by shoring from the upper floors in multi-story construction, the need for shoring of the non-composite section becomes even more critical.
9. Stub-girders should be fabricated or shored to provide a camber that is equal to the dead load deflection of the member.

10. The overall strength of a stub-girder is not controlled by the compressive strength of the concrete slab, therefore the use of high-strength concrete mixes provides no advantage.
11. It is typical for the ribs of the metal floor deck to run parallel to the span of the stub-girder. This orientation of the ribs therefore increases the area of the top chord slab and also makes it possible to arrange a continuous rib or trough directly above the stubs, which in turn improves the composite interaction of the slab with the stub-girder.
12. Welds between the bottom of the stubs and the top of the continuous bottom beam should be concentrated at the ends of the stubs where the forces between these two elements are the greatest.
13. Internal longitudinal slab reinforcement to add strength, ductility and stiffness to the stub-girder should be provided in two layers, one just below and one just above the heads of the shear studs.
14. Internal transverse slab reinforcement should be provided to add shear strength and ductility. Placing the transverse reinforcement in a herring bond pattern - i.e., diagonal to the direction of the stub-girder span - will also increase the effective width of the concrete flange/top chord.
15. The flexural stiffness of the top chord slab of a stub-girder should be based on the conventional effective width allowed by standard composite beam design criteria, except that the transformed section should include the contribution of both the metal deck and the internal longitudinal reinforcement.
16. It is not proper to include the top flange of the stubs in the calculation of the moment of inertia of the top chord slab element.
17. Modeling of the stubs as the verticals of the Vierendeel truss/girder involves dividing the stubs up into vertical elements equal to one-foot lengths of the section spaced at one foot on center from one end of the stub to the other. The vertical stub elements should be modeled as fixed at the top and bottom, at the top chord (concrete slab) and bottom chord (continuous beam) of the truss/girder, respectively.
18. The transverse floor beams should be modeled as a single vertical web member/element of the truss/girder. The top and bottom of the member should be modeled as pinned at the top and bottom chords.

In conclusion, it can be stated that the stub-girder method of construction was and still is an innovative solution to multi-story, framed steel floor construction. However, as deeper wide flange sections became more available in the marketplace and design engineers became more accustomed to analyzing web holes in wide flange beams, the use of stub-girder construction waned.

In addition, because of the extra labor costs associated with the fabrication of stub-girders and the necessity to construct stub-girders as shored composite construction, the system priced itself out of the industry.

Canadian Test of Stub Girder
Source: Ultimate Strength Analysis of Stub Girders

Post-Tensioned Concrete Construction

This method of construction is still very much in use today. Therefore, this presentation will provide a history of how the post-tensioning industry developed in the United States.

Two-Way Post-Tensioned (PT) Beams

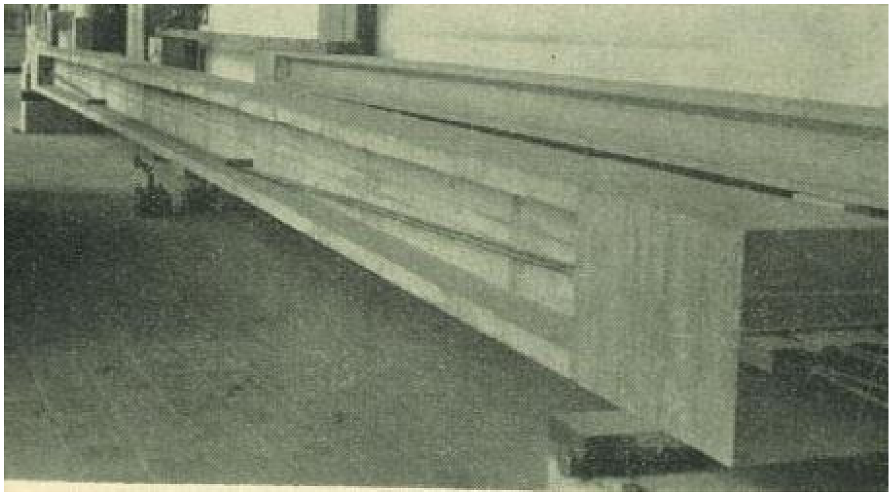
Source: PicsToPin

When writing each article for this series, I have always strived to use my own words, or at least paraphrase the material that was available for each system discussed. However, during the development of this article on post-tensioned concrete construction, it was necessary to follow closely and sometimes quote specific sections from the following source; "Post-Tensioned Concrete in Buildings, Past and Future, an Insiders View," by Kenneth B. Bondy, PTI Journal, Vol. 4, No.2, December 2006. The reason for this approach is as follows.

Most of the articles for this series required the compiling of information from a number of different sources as indicated by the volume of references noted at the beginning of this course. However, in the case of the history of post-tensioned concrete, Mr. Bondy was really the only available source of information concerning the development of this method of construction in the U.S. In addition, because Mr. Bondy's presentation of the chronology and innovations of the post-tensioned industry was based on his own personal experiences, his paper that was published by PTI in 2006 served as the definitive resource on the subject. As a result, the article that I wrote could not improve on what Mr. Bondy presented; consequently, I chose to follow the same basic outline as his 2006 paper and sometimes quote specific sections from his paper.

In the end, it was my hope that the original published article served the purpose of compiling and disseminating information concerning existing structural systems and ultimately highlighted the best source of further information on the history of the development of post-tensioned concrete construction, which is Mr. Bondy's 2006 paper. As a result of all of the above, all information for this segment of the course can be found at:

http://www.kenbondy.com/images/ProfessionalArticles/Bondy%20Dec%202006.pdf



Externally Post-Tensioned Beam

Source: Prestressed Concrete, 2nd Edition

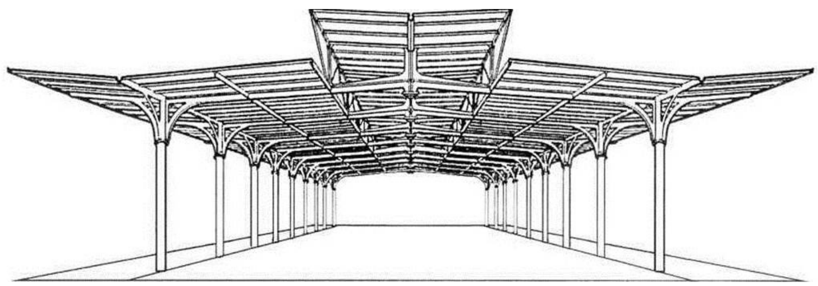
Conclusions

It is anticipated that the post-tensioning industry will continue to thrive world wide over the near foreseeable future for both building and bridge construction. It is also anticipated that further advancements within the industry will continue to enhance the design and construction of post-tensioned structures. In fact, the author anticipates that sometime in the future an advanced chemical sheathing will be developed for mono-strand construction that will allow for conventional unbonded installation and tensioning yet subsequently create a long-term bonded condition in the absence of internal ducts or grouting. (**Source of Image:** PicsToPin)

Wrought and Cast Iron

Ferrous metals used in construction can be categorized into three principal iron-carbon alloys, based on approximate carbon content: wrought iron (0.020% to 0.035%), steel (0.06% to 2.00%) and cast iron (2% to 4%). Wrought iron is almost pure iron and contains between 1% to 4% slag (iron silicate). The slag is not alloyed into the wrought iron, which gives the material its characteristic laminated (or layered) fibrous appearance. Wrought iron can also be distinguished from cast iron by its generally simpler forms and less uniform appearance.

Cast iron contains varying amounts of silicon, sulfur, manganese and phosphorus. Cast iron, while molten, is easily poured into sand molds, making it possible to create unlimited forms which also results in mold lines, flaws and air holes. Cast iron elements are commonly bolted or screwed together, while wrought iron is either riveted or welded. (Unless noted otherwise, all images of iron members or structures have reprinted with the permission of Margot Gayle)



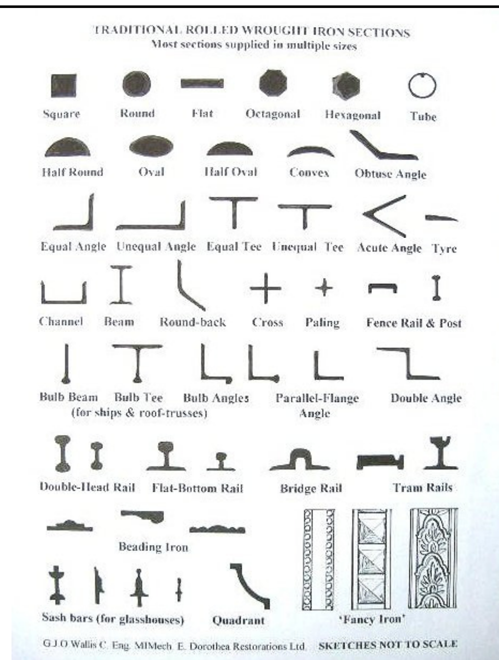
Iron Structure

Wrought Iron

Wrought iron refers to ferrous metals that can be worked or "wrought" on an anvil or shaped and forged in rolling machines. Wrought iron is tough and stringy and has an elasticity that was conducive for use in bolts, beams and built-up girders. Wrought iron is also easily welded.

Until the mid-1800's, wrought iron in buildings was used primarily for tie rods, straps, nails, hardware or decorative railing and balconies. Around 1850, the structural use of wrought iron became more prevalent as rail beams, bulb-tees, channels and I-beams became commercially available.

When wrought iron was employed as tie rods, the material was typically used in conjunction with cast iron anchor plates shaped as stars, rosettes or S's. In built-up girders, wrought iron was also used in conjunction with cast iron. Initially, these composite built-up girders were constructed as bowstring trusses with an upper cast iron chord and a lower wrought iron tie rod (See Figures 1a and 1b on the next two slides).



Source: Dorothea Restorations

This a drawing of a composite cast iron and wrought iron bow string truss.

The truss supports a brick arch and wrought iron rail beam floor.

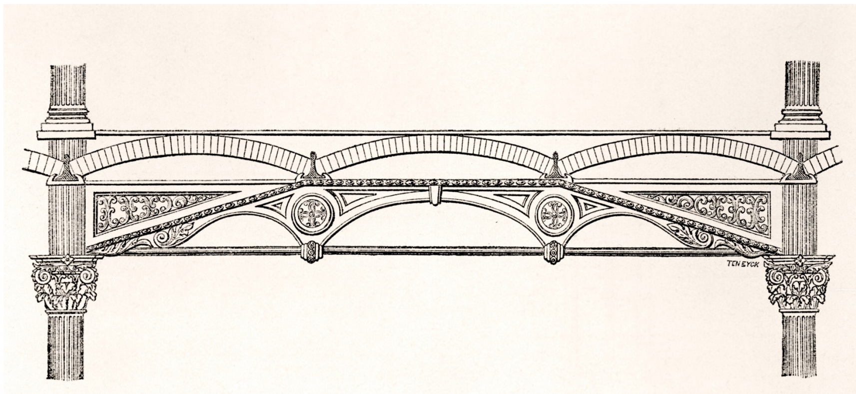


Figure 1a

Source: Peabody Essex Museum

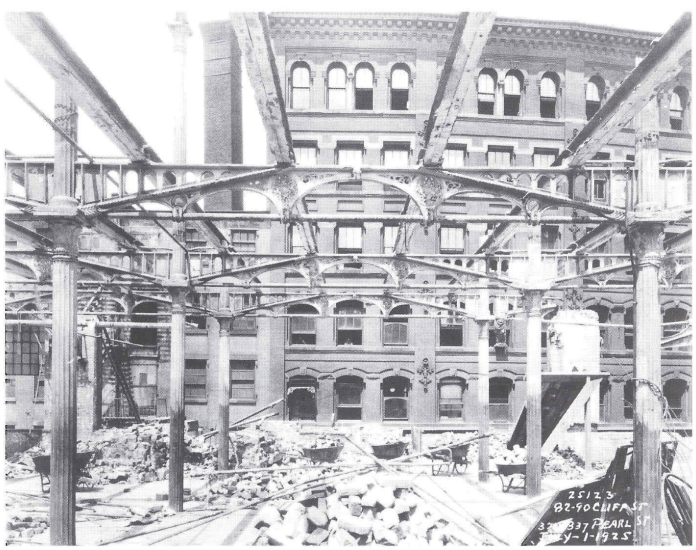


Figure 1b

Source: Alan Burnham Archive

Later versions of composite construction included perforated girders generally constructed with cast iron in the top three-quarters of the member and wrought iron in the bottom portion (Figure 2).

Figure 2

Source: Historic American Building Survey

In 1854, the Trenton Iron Works manufactured the first rolled wrought iron beam in the U.S. (a 7-inch-deep, bulb-tee rail beam). The following year, the Trenton Iron Works also manufactured the first I-beam in the U.S. (the "Cooper beam"). I-beams had previously been rolled commercially in France in the 1840's. Prior to the development of the bulb-tee rail beam, wrought iron deck beams had been rolled for use in the shipbuilding industry, and prior to the development of the Cooper beam, channel beams had been manufactured. Figure 3 shows the evolution of all of these rolled wrought iron sections.

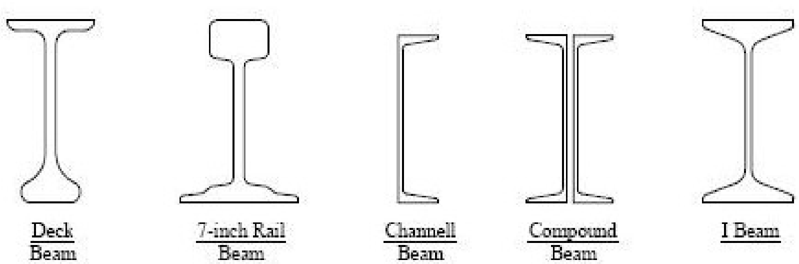
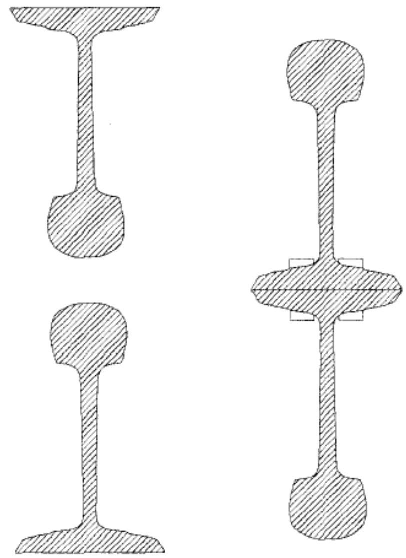


FIGURE 3



Additional Examples of Rail Beams

Source: U.S. Government Printing Office

Rolled wrought iron beams continued to be used for several decades after the mid 1850's, even after structural steel became available. However, as the quality and quantity of available steel improved, the use of wrought iron gradually came to an end. In general, the use of wrought iron beam framing in conjunction with cast iron compression components lasted from the mid-1850's until the late 1890's.

Home Insurance Building Chicago

Source: Library of Congress

The earliest known tabulation of wrought iron structural shapes was published by Carnegie Kloman and Company in 1873. The largest beam listed was a 15-inch-deep I-beam that weighed 67 pounds per foot.

This publication is very rare, but an 1876 edition is available in the Library of Congress. The author was able to locate a digitized copy of the 1892 8th Edition of the Wrought Iron and Steel Construction manual published by Pencoyd Iron Works of Philadelphia on Google Book Search.

8″ IRON I BEAMS.—No. 11.

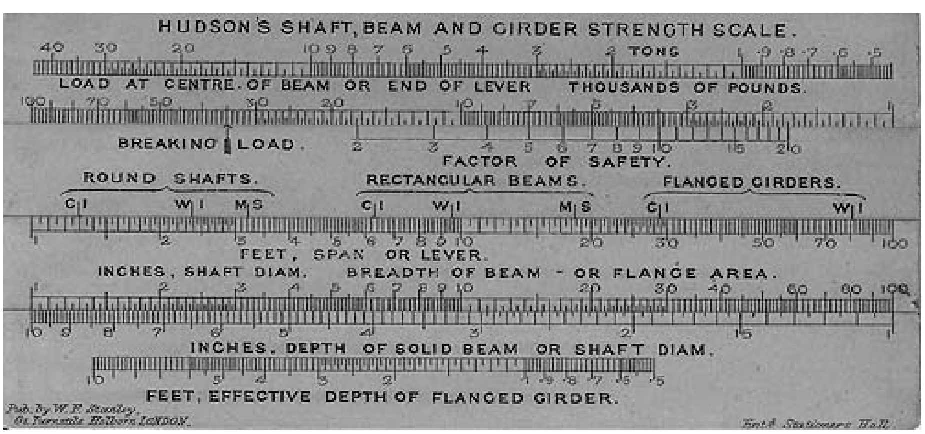
LEAST SECTION.		GREATEST SECTION.	
Flange width,	4.33	Flange width,	4.57
Web thickness,	.41	Web thickness,	.60
Area in square inches,	8.26	Area in square inches,	9.78
Resistance,	21.20	Resistance,	23.23
Pounds per foot,	27.53	Pounds per foot,	32.60

Greatest safe load in net tons evenly distributed, including beam itself.
For a load in middle of beam, allow one-half of the tabular load.
Deflection for centre load will be $\frac{8}{10}$ of the tabular deflection.
Figures in small type denote cases where deflection is excessive

Distance Between Supports in Feet.	Greatest Safe Load in Net Tons for Least Section.	Addition to Safe Load for Each Pound per Foot Increase.	Deflection in Inches.	Greatest Distance in Feet Between Centres of Beams of Least Section for Distributed Loads as Below. 100 Pounds per Sq. Foot.	150 Pounds per Sq. Foot.	200 Pounds per Sq. Foot.	250 Pounds per Sq. Foot.	Divide by Load per Sq. Foot and Add to Corresponding Distance for Each Pound per Foot Increase of Beam.
6	16.49	.31	.07	54.97	36.64	27.48	21.99	103.70
7	14.13	.27	.09	40.37	26.91	20.18	16.15	76.18
8	12.37	.23	.12	30.93	20.62	15.46	12.37	58.33
9	10.99	.21	.15	24.42	16.28	12.21	9.77	46.09
10	9.89	.19	.18	19.78	13.19	9.89	7.91	37.33
11	8.99	.17	.22	16.35	10.90	8.17	6.54	30.85
12	8.24	.16	.26	13.73	9.16	6.87	5.49	25.92
13	7.61	.14	.31	11.71	7.81	5.85	4.68	22.09
14	7.07	.13	.36	10.10	6.73	5.05	4.04	19.05
15	6.60	.12	.41	8.80	5.87	4.40	3.52	16.59
16	6.18	.12	.47	7.73	5.15	3.86	3.09	14.58
17	5.82	.11	.53	6.85	4.56	3.42	2.74	12.92
18	5.50	.10	.59	6.11	4.07	3.06	2.44	11.52
19	5.21	.10	.66	5.48	3.66	2.74	2.19	10.34
20	4.95	.09	.73	4.95	3.30	2.48	1.98	9.33
21	4.71	.09	.81	4.49	2.99	2.24	1.79	8.46
22	4.50	.08	.89	4.09	2.73	2.05	1.64	7.71
23	4.30	.08	.97	3.74	2.49	1.87	1.50	7.06
24	4.12	.08	1.06	3.43	2.29	1.72	1.37	6.48
25	3.96	.07	1.15	3.17	2.11	1.58	1.27	5.97
26	3.80	.07	1.24	2.92	1.95	1.46	1.17	5.52
27	3.66	.07	1.34	2.71	1.81	1.36	1.08	5.12
28	3.53	.07	1.44	2.52	1.68	1.26	1.01	4.76
29	3.41	.06	1.54	2.35	1.57	1.18	0.94	4.44

Example of Pencoyd Manual

Material and design properties provided in the Pencoyd Iron Works manual indicate that the ultimate tensile strength of wrought iron was as high as 50,000 psi. The allowable extreme fiber stress was indicated as 14,000 psi for wrought iron and 16,800 for steel. Limits for safe loads on wrought iron beams provided in the design tables include the character of the service load (i.e. static, fluctuating or impact) and extent of lateral support.



Hudson's Shaft, Beam and Girder Scale
(used for determining structures sizes of cast iron and wrought iron members)

Source: Peter Fox

Cast Iron

Cast iron is very hard and resists compression forces very well, but because of the carbon content, it is also very brittle. Because of this, cast iron was used primarily for the construction of columns and compression elements of composite wrought iron girders. Shortly before the 1800's, cast iron was used for columns in multi-story, wood-framed factory buildings in England. Cast iron was used because of its strength and perceived fire resistance. The combination of cast iron columns and wood framing continued to be widely used for the next half century; however, by the mid-1800's, wrought iron beams began to replace wood.

Cast-Iron Arches

Cast iron used for structural purposes during the 1800's typically had a compressive strength of 80,000 psi.

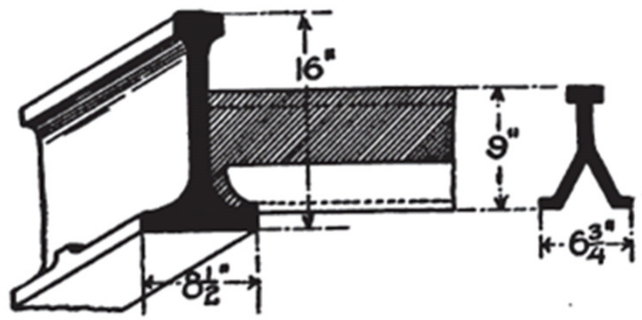
Although there was no clearly defined yield point of the material, the tensile strength ranged between 10,000 and 15,000 psi.

The brittleness of the material, in conjunction with its inherent manufacturing flaws, made cast iron highly susceptible to tensile failures; therefore, for most of the 1800's, cast iron was only used to resist compression forces.

Palm House at Kew Gardens London

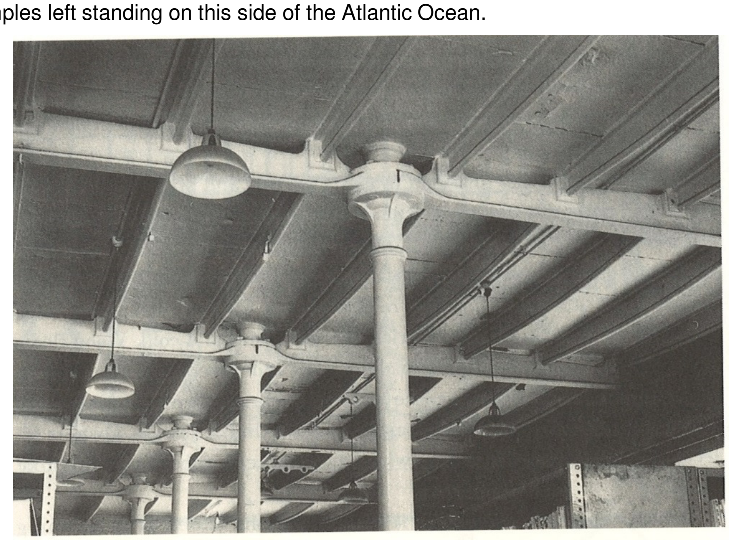
Source: GreatBuildings.com

Cast iron was also used for floor framing as illustrated in this slide. This system of cast iron floor framing was used in the Boston Public Library in the 1850's and included inverted T girders spaced at approximately 10-feet on center that supported inverted Y purlins spaced at approximately 4-feet on center. The Y purlins supported a brick arch floor system. Inverted T configurations of cast iron beams were common after it was determined in the 1840's that non-symmetrical cross sections with a 7 to 1 ratio of the bottom to top flange cross sections was the most efficient arrangement for cast iron.



Scope: Freitag

Here is another interesting all cast iron method of floor framing, expect this system used pieces of flagstone to form the flooring between the purlins and beams. Although this system originated in England it was used in the U.S., however, there are no surviving examples left standing on this side of the Atlantic Ocean.



Source: Institution of Civil Engineers

Cast-Iron Column to Wrought Iron Beam Connection

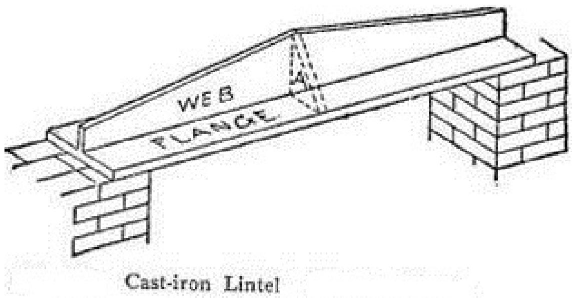
In the mid-1800's, cast iron began to be used for the construction of the front façades of buildings (Figure 4). The cast iron façade was used for both decorative and structural purposes, as the adjacent floor framing was supported by the cast iron.

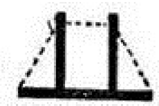
Figure 4

Cast Iron Façade Example
E.V. Haughwout Building – New York City
Source: Steve Guttman

Cast iron was also used in conjunction with masonry façades as exposed decorative/structural window and door lintels (Figure 5), as well as for balconies and verandas. Many of these types of exterior structures still exist today, thanks to ongoing painting and maintenance efforts.

Figure 5



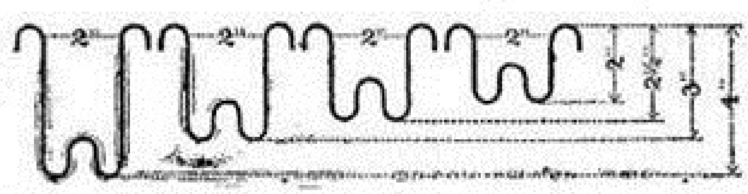


Cast Iron Lintels

Source: Building Construction and Superintendence

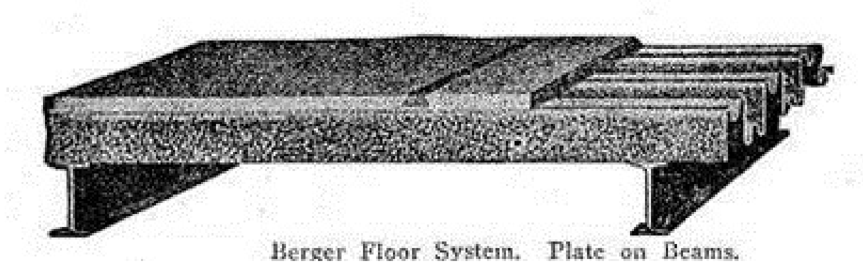
Rolled Sheet and Corrugated Iron

The first sheet iron in the U.S. was rolled in Trenton, New Jersey in the late 1700's. Sheet iron was used as a flooring and roofing material up until the end of the 1800's. Corrugation of sheet iron, patented in England in 1829, was first used in the U.S in the 1830's. Corrugated sheet iron was typically painted with pitch, which was later replaced by galvanizing. Corrugated iron was also manufactured in arched sheets for floor construction. Typically, the ends of the arched sheets were supported by the bottom flanges of wrought iron I-beams, with concrete then cast on top of the sheets. This type of construction replaced brick arch floor construction, which had been used extensively with wrought iron I-beams previously.



Berger Corrugated Steel Plate

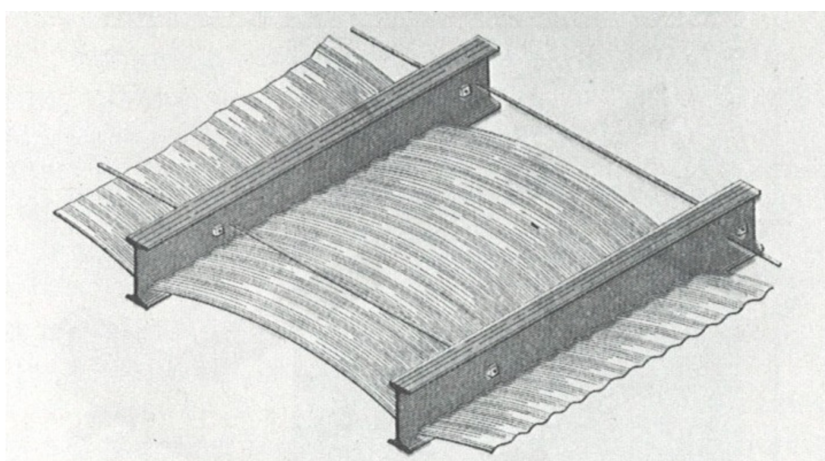
Source: Building Construction and Superintendence



Berger Corrugated Steel Floor Plate System

Source: Building Construction and Superintendence

Corrugated iron sheets were also used for arched floor construction starting in the late 1860's. The space above the sheets was filled with concrete encasing the beams and tie-rods.



Source: Library of Congress

Source: Sara Wermiel

Building Construction Overview

The use of prefabricated cast iron façades, wrought iron beams and cast iron beam and column components served as a forerunner to the multi-story, steel-framed buildings of today.

In addition, specialty wrought iron structures constructed in New York City in the mid 1850's such as firewatch towers and gunshot manufacturing towers that employed drilled and socketed rock foundation anchors and infill masonry walls served as a precursor to early steel skyscraper construction.

It is also interesting to note that the manufacturers of cast iron façades and other exposed components often identified the source of the material by either affixing foundry labels to the building or by using trademarks in the ornamentation of the decorative portions of the façade. This information can sometimes be used to assist in the structural evaluation of an existing building.

The Tower Building - NYC

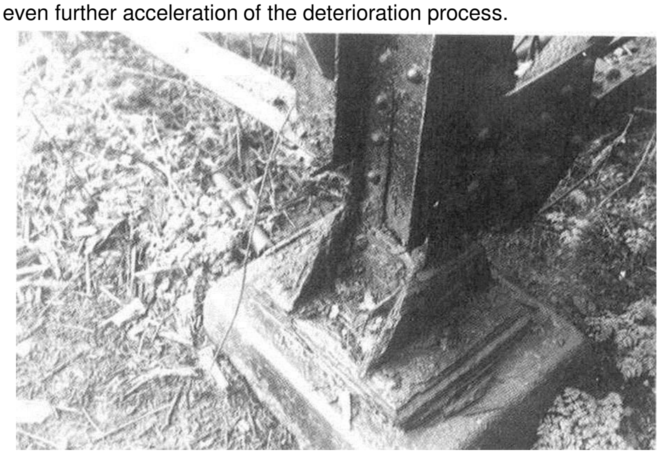
Source: www.nyc-architecture.com

This slide illustrates an example of an angle bracket at each end of a beam that was used to form a rigid frame in a 12-story tower. Both the steel beam and brackets have been encased on concrete.

National State Bank Building in Newark, New Jersey

Deterioration

Iron oxidizes rapidly when exposed to moisture and air. The product of the oxidation process is rust. The minimum relative humidity to promote "rusting" is 65%, but humidity levels lower than this can cause oxidization in the presence of pollutants. In addition, if chlorides are present, the corrosion process can become accelerated. Once a film of rust starts to develop, the natural porosity of the corrosion byproduct tends to act as a reservoir for moisture, resulting in even further acceleration of the deterioration process.

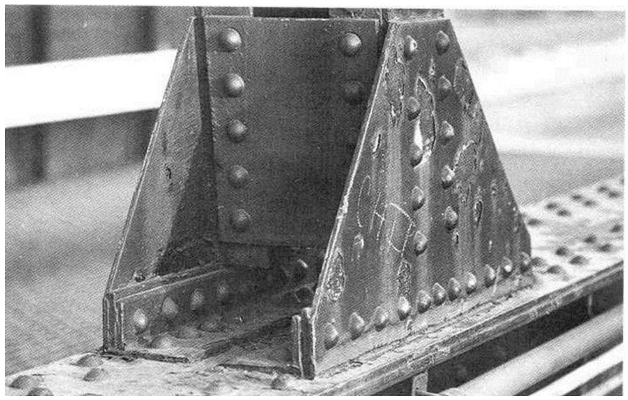


Wrought Iron Deterioration

Source: John G. Waite

Cast iron will develop a somewhat protective surface scale, which makes it slightly more resistant to corrosion than wrought iron; however, it is still recommended that cast iron be painted to prevent rusting. Iron can also be corroded by acids, magnesium and some sulfur compounds.

Dissimilar metal galvanic corrosion can also occur between iron and copper, chromium, lead, stainless steel and brass. In general, wrought iron rusts more rapidly than cast iron. However, because of the slag content of wrought iron, the material is more resistant to progressive corrosion than cast iron.



Example of Minor Deterioration

Source: John G. Waite

Repair and Restoration

There are a number of methods available to remove paint and corrosion from cast and wrought iron, including manual scraping, chipping or wire brushing; low-pressure grit or sandblasting; flame cutting; and chemical removal. Once any existing paint and corrosion have been removed, the most common method of protecting cast iron from further deterioration is to repaint the surface.

Prior to repainting, it is first necessary to prepare or repair the surface. Proper preparation includes elimination of crevices or pockets that can collect moisture, to prevent accelerated deterioration; removal or smoothing of sharp corners, to prevent accelerated paint failure; hermetical sealing of hollow section, to prevent moisture intrusion and freeze/thaw damage; filling of joints, cracks and bolt or screw holes with sealant, to prevent moisture intrusion and freeze/thaw damage; and following the paint manufacturer’s specifications.

Another type of deterioration common to cast iron is graphitization. This condition can occur in the presence of acidic precipitation or seawater. As the iron corrodes because of exposure to these types of environments, porous graphite residue is impregnated within the surface corrosion byproduct. The cast iron element retains its original appearance and shape, but becomes gradually weaker internally.

Graphitization typically occurs when cast iron is not painted for extended periods or where the sealant has failed at the joints between adjoining components. This condition can be identified in the field by scraping the surface of the cast iron with a knife to see if deterioration of the iron is revealed beneath the surface.

In all cases, it is recommended that a test area be used to confirm that the selected cleaning, preparation and painting techniques are effective prior to attempting to remediate the entire restoration area. It is also recommended that sheltered areas such as eaves, where evaporation of moisture can be inhibited, be coated with additional layers of the selected paint or coating.

Additional protection and repair procedures can also include plating with metals or cladding with plastic or epoxy. Sections or entire portions of a significantly deteriorated area may be replaced with glass fiber reinforced concrete (GFRC), fiber reinforced polyester (FRP) or aluminum.

If additional structural retrofitting is required as a part of an adaptive reuse project, the following remedial work should be avoided if at all possible: welding, burning of holes, the use of impact drills, high strength bolts, and filling of voids or posts with concrete.

Open Web Steel Joists

This method of construction that is still very much in use today. Never the less, the historic, original construction practices described in this presentation may still be encountered in existing structures.

Open Web Joists & Joist Girders

Source: Electronic Library of Construction Occupational Safety & Health

Part I: History

I would like to thank the Steel Joist Institute (SJI) for providing much of the material that was used in the development of this article. In fact a brief history of open web joists is provided in the Catalog of Standard Specifications and Load Tables for Steel Joists and Joist Girders published by the SJI. A brief summary of this history is as follows:

1923 The first warren type, open web truss/joist is manufactured using continuous round bars for the top and bottom chords with a continuous bent round bar used for the web members.

1928 First standard specifications adopted after the formation of the SJI. This initial type of open web steel joists was later identified as the SJ-Series.

1929 First load table published.

1953 Introduction of the longspan or L-Series joists for spans up to 96 feet with depths of up to 48 inches, which were jointly approved by the AISC.

1959 Introduction of the S-Series joists which replaced the SJ-Series joists. The allowable tensile strength was increased from 18 ksi to 20 ksi and joist depths and spans were increased to 24 inches and 48 feet, respectively.

1961 Introduction of the J-Series joists which replaced the S-Series joists. The allowable tensile strength was increased from 20 ksi to 22 ksi. Introduction of the LA-Series joists to replace the L-Series joists which included an allowable tensile strength increase from 20 ksi to 22 ksi. Introduction of the H-Series joists which provided an allowable tensile strength of 30 ksi.

1962 Introduction of the LH-Series joists which provided yield strengths between 36 ksi and 50 ksi.

1965 Development of a single specification for the J and H-Series joists by the SJI and AISC.

1966 Introduction of the LJ-Series joists which replaced the LA-Series joist. In addition, a single specification was developed for the LJ and LH-Series joists.

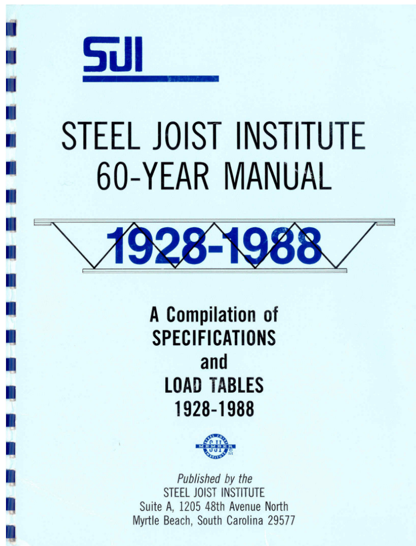
1970 Introduction of the DLH and DLJ Series joists which included depths up to 72 inches and spans up to 144 feet.

1978 Introduction of Joist Girders including standard specifications and weight tables.

1986 Introduction of the K-Series joists which replaced the H-Series joists.

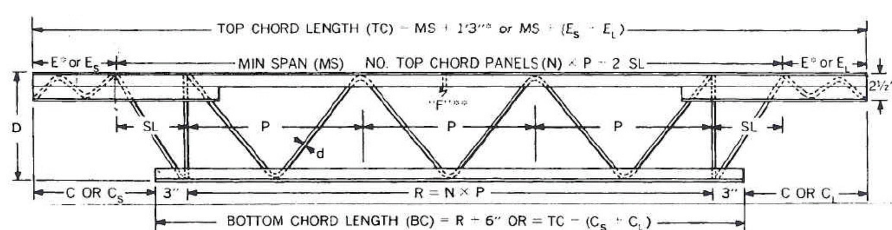
1994 Introduction of the KCS joists which provided a constant moment and shear capacity envelope across the entire length of the member.

The SJI also publishes a 75-Year Manual which includes a complete chronological listing of the standard specifications and load tables for all of the steel joists and weight tables for the Joist Girders previously made available by the SJI over the time period from 1928 to 2005. This publication can be an invaluable tool for an engineer involved in the analysis of existing buildings constructed with open web steel joists.



In addition to the steel joists presented in SJI 75-Year Manual, there were also a number of joists produced by manufacturers that were either never members or later joined the SJI.

Some of these manufacturers include: Ashland Steel Joists (manufactured by Ashland Steel Products Co., Inc – Ashland City, Tennessee); Vescom Structural Systems, Inc. – Westbury, New York; Ridgeway Joists (manufactured by Continental Steel Ltd. – Coquitlam, British Columbia); Northwest Joist Limited (a Division of Brittain Steel Limited – New Westminster, British Columbia); Cadmus Long Span and Joist Corporation (affiliated with Alexandria Iron Works, Inc. – Alexandria, Virginia); T-Chord Longspan Joists (manufactured by the Haven Busch Company – Grandville and Grand Rapids, Michigan); and the Macomber Steel Company – Canton, Ohio. Table 1 provides a summary description of the joists produced by these same manufacturers.



*E = 7½" unless E_S and E_L specified. (NOTE: 2 E = 1'3")

**"F" Fillers having same diameter as web bar are spaced in top chord panels between quarter span points.

Source: SJI

TABLE 1

Unique Open Web Joists
(Load Tables may be available from SJI)

System	Figure	Description	Yield Strength	Depth (inches)	Span (feet)	Chords	Webs	Notes
Ashland	1	HS-Series Joists	50 ksi	8 to 24	8 to 48	Double angles	Round bars	
	N/A	LS-Series Joists	50 ksi	Unknown	64 maximum	Unknown	Unknown	
Cadmus	5	1952 Structural T Longspan & Standard Joists	See Note 6	10± to 54	12'-6" to 108	Split T	Angles	6, 7
Haven Busch	6	1952 to 1962 T-Chord Long-span Joists	See Note 9	18 to 88	25 to 175	Split T	Angles	8, 9
Macomber	7	Purlin or Steel Joist	Unknown	8 to 16	10 to 26	See Note 10	Round bars	10
	8	Massillon Steel Joist	Unknown	8 to 16	4 to 31	Round bars	Round bars	
	9	Canton Steel Joist	Unknown	8 to 16	Unknown	Double angles	Round bars	
	10	Buffalo Steel Joist	Unknown	8 to 16	Unknown	See Note 11	Round bars	11
	N/A	Special Joists	Unknown	12 to 20	8 to 40	Unknown	Unknown	
	11	Residence Joist	Unknown	6 to 10	6 to 20	See Note 12	Round bars	12
	12	Standard Longspan Joist	See Note 14	18 to 40	24 to 72	Double angles	Angles & bars	13, 14
	N/A	Intermediate Longspan	See Note 14	18 to 22	20 to 44	See Note 10	Round bars	10, 14
	13	1955 New Yorker	Unknown	8 to 24	7 to 48	V shaped plates	Round bars	
	14	V or Double V Bar Joist	Unknown	8 to 22	4 to 44	V shaped plates	Round bars	
	N/A	V-Girders	Unknown	18 to 48	13 to 96	V shaped plates	Round bars	
	15	V-Purlin	Unknown	8 to 60	8 to 120	V shaped plates	See Note 15	15
	16	Allspan	Unknown	8 to 76	8 to 152	V & Double V shaped plates	See Note 15	15
	N/A	V-Lok Purlin	Unknown	8 to 36	8 to 72	V & Double V shaped plates	Round bars or round pipes	16, 17
	N/A	V-Lok Girder	Unknown	12 to 40	15 to 50	See Note 18	Round bars or Angles	16, 18
	18	V-Beam	Unknown	8 to 28	8 to 56	See note 19	Round bars	19
Northwest	4	Series 1, 2, 3 & 4 Joists	See Note 5	12 to 72	12 to 80	V shaped plates	Square bars & round pipes	4, 5
Ridgeway	3	Open Web Joists	See Note 3	12± to 47±	16± to 59±	V shaped plates	Square bars & round pipes	3
Vescom	2	Composite Floor Joists	36 & 50 ksi	8 to 40	20 to 48	Double angles	Round bars	1
	N/A	Composite Truss Girders	36 & 50 ksi	16 to 40	20 to 50	Double angles	Angles	2

Notes:

1. Top chord included deformed, extended vertical leg of one angle for composite action with surrounding concrete slab.
2. Top chord included deformed, extended vertical plate in addition to double angles for composite action with surrounding concrete slab.
3. Web allowable stress: 36 ksi (bars) & 50 ksi (pipes); Chord allowable stress: 54 ksi.
4. Joist designs over 80 feet spans were available upon request.
5. Web allowable stress: 33 & 44 ksi (bars), 50 ksi (pipes); Chord allowable stress: 55 ksi.
6. Allowable compressive stress for top chord or web members = 15 ksi. Allowable combined compressive stress at top chord panel points and allowable tensile stress = 18 ksi.
7. Chord tees cut from standard wide flange or junior beams.
8. Available as parallel chord, single or double sloped top chord or hipped end configurations.
9. Allowable combined compressive stress at mid-panel chord and web = 15 ksi (1952); 20 ksi (1956). Allowable combined compressive stress at panel points = 24 ksi (1956). Allowable tensile stress = 20 ksi (1952 & 1956).
10. Double angle top chord; Round bars bottom chord.
11. Inverted double angle top chord; Round bars bottom chord.
12. Single steel angle and wood nailer top chord; Round bars bottom.
13. Available as parallel chord or single or double sloped top chord.
14. Allowable combined direct and bending stress in top chords = 20 ksi.
15. Sizes #2 - #9: Round bars; Sizes #10 up through #22: Angles.
16. Included proprietary stud and slot end bearing connection – See Figure 17.
17. Round bars, round pipes or angles.
18. V & double V shaped plates or double angles.
19. V shaped top chord & U shaped bottom chord plates.

In addition, some manufacturers, prior to becoming SJI members produced products other than the historical standard SJI joists series. Some of these manufacturers include: Truscon Steel Company – Youngstown, Ohio; Macmar and Kalmantruss joists (manufactured by Kalman Steel Corporation, a Subsidiary of Bethlehem Steel Company – Bethlehem, Pennsylvania); and Gabriel Steel Company - Table 2 provides a summary description of the joists produced by these same manufacturers. Detroit, Michigan. In addition to the information provided in Table 2, it should be noted that Bethlehem Steel Company also produced cold formed joists with hat channel sections for the chord members, and Gabriel Steel Company also produced unique V shaped top chord and single round bar bottom chord members. (**Source of Image:** SJI)

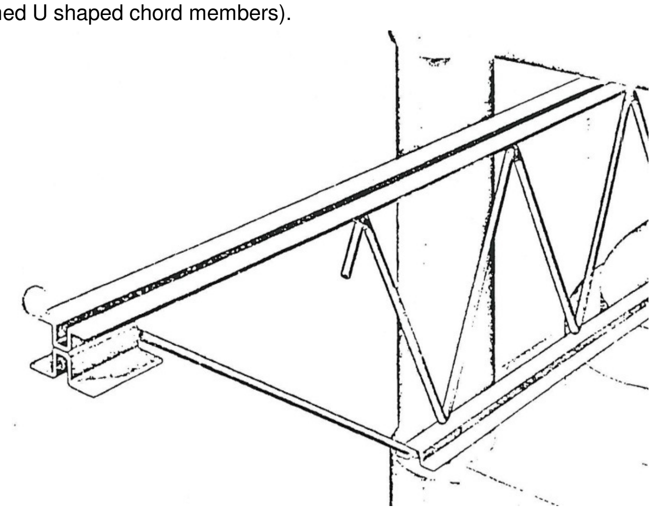
TABLE 2

Unique Open Web Joists (Load Tables may be available from SJI)								
System	**Figure**	**Description**	**Yield Strength**	**Depth (inches)**	**Span (feet)**	**Chords**	**Webs**	**Notes**
Bethlehem	24	KalmanTruss Joists	See Note 8	8 to 16	4 to 32	T shape	Rectangular	7, 8, 9
	25	MacMar Joists	See Note 10	8 to 16	4 to 32	Angles	Round bars	10
	26	BLJ Series	See Note 11	52 to 60	89 to 120	Structural Tee	Angles	11
	26	BLH Series	See Note 12	52 to 60	89 to 120	Structural Tee	Angles	12
	27	Standard Open Web Joist	See Note 13	8 to 16	4 to 32	Angles	Round bars	13
	28	Longspan Open Web Joist	See Note 14	18 to 32	25 to 64	Angles	Angles	4, 14
	29	BJ Series	See Note 11	24 to 30	24 to 60	See Note 15	Round bars	11, 15
	29	BH Series	See Note 12	24 to 30	24 to 60	See Note 15	Round bars	12, 15
Gabriel	30	Long Span Joist		18 to 32	24 to 64	Angles	Round bars	4
Truscon	19 & 20	O-T (Open Truss) Joists	See Note 1	8 to 20	7 to 40	"Tee" & M shaped plates	Round bars	1
	21	Series AS Joists	See Note 2	8 to 24	7 to 48	U shaped	Round bars	2
	21	Series BB Joists	See Note 3	8 to 24	7 to 48	U shaped	Round bars	3
	22 & 23	Clerespan Joists	See Note 6	18 to 32	26 to 64	"Tee" & angles	Angles & bars	4, 5, 6

Notes:

1. Web allowable stress: 19,000 psi - 100(l/r); Chord allowable stress: 16,000 psi.
2. Cold formed chord allowable tension: 25 ksi; Hot rolled web members allowable compression: 17,000 psi - 100(l/r).
3. Cold formed chord allowable tension: 28.5 ksi; Hot rolled web members allowable compression: 19,000 psi - 100(l/r).
4. Available as parallel chord, single or double sloped top chord configurations.
5. Chord angles were some times arranged toe to toe for channel configuration.
6. Allowable combined top chord compressive stress: 15 ksi; Allowable bottom chord tensile stress: 18 ksi.
7. Manufactured by punching web opening in blanks such that chords and webs do not have to be welded together.
8. Allowable tensile stress: 16 and 18 ksi.
9. Also marked as Kalman Joist.
10. Allowable tensile stress: 18 ksi.
11. Maximum tensile working stress: 22 ksi.
12. Maximum tensile working stress: 30 ksi.
13. Design tensile stress: 18 ksi.
14. Allowable combined compressive stress at panel points and allowable tensile stress = 18 ksi. Allowable combined compressive stress at mid-panel and compression webs = 15 ksi.
15. Double angle top chord; Round bars bottom chord.

Additional manufacturers not included in Tables 1 and 2 include: Berger Steel Company (double V shaped chord members); Armco Steel (cold formed hat channel chord members); Raychord Corporation (cold formed hat channel and U shaped chord members); Republic Steel (cold formed hat channel chord members); and USS AmBridge (cold formed U shaped chord members).

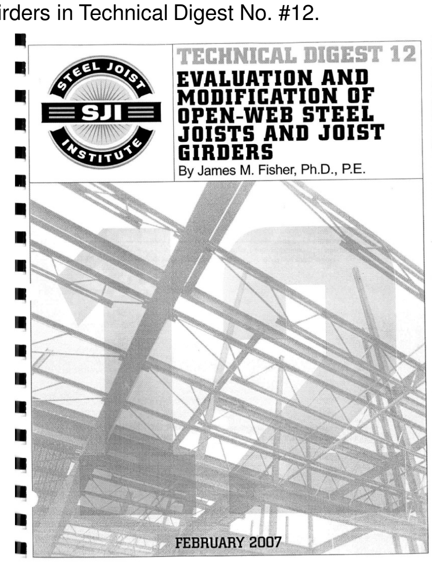


Source: Truscon Steel Co.

**Example of Locally Fabricated Riveted Joist
from the Chester County Courthouse, PA**

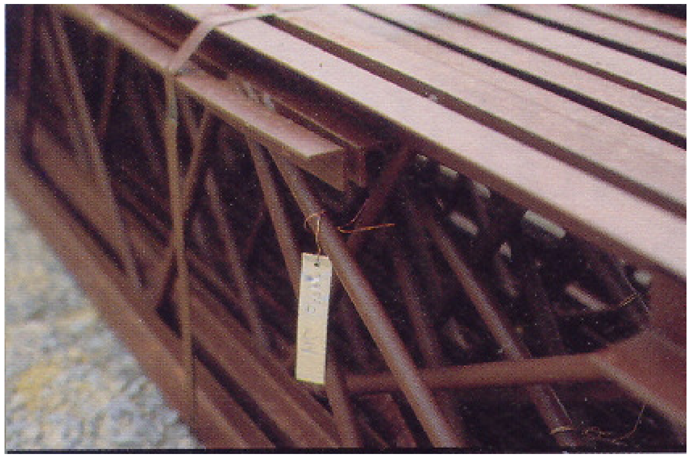
<u>Part II: Evaluation and Modification of Existing Joists</u>

The evaluation and strengthening of existing open web steel joists and Joist Girders is often required as a result of equipment upgrades or new installations and adaptive or change in use of a facility. The SJI provides an excellent resource for the evaluation and modification of existing joists and joist girders in Technical Digest No. #12.



The first step in the process of evaluating an existing joist is to determine the capacity of the member. Ideally, the best method of determining the member capacity is through the original construction or shop drawings which allow the identity of the joist to be established.

Similarly, it is also sometimes possible to identify the joist via fabrication tags left attached to the joists in the field. However, if tags can be found, more often than not the tag only identifies the shop piece mark number rather than the actual joist designation.



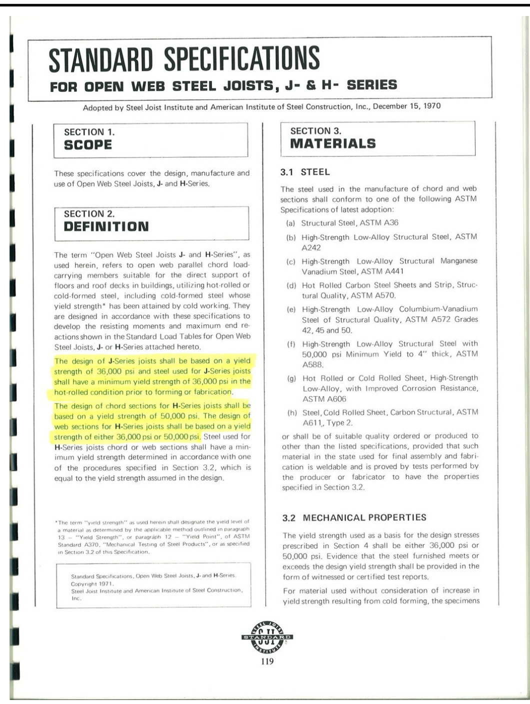
Source: National Band & Tag Co.

In some instances, it may only be possible to establish the type or series of the joist through the available documentation.

In this situation it is possible to conservatively assume that the capacity of the existing joist is no more than the lightest joist in the series for the given depth.

In addition, if it is not clear whether a J or an H-Series joist is involved, the J-Series joist should always be conservatively assumed because of its lower load carrying capacity.

However, if a definitive distinction is required, and it is possible to secure a material sample in order to obtain results from a standard ASTM tension coupon test, a determination as to whether the joist is 36 ksi (J-Series) or 50 ksi (H-Series) can be made.



STANDARD SPECIFICATIONS
FOR OPEN WEB STEEL JOISTS, J- & H- SERIES

Adopted by Steel Joist Institute and American Institute of Steel Construction, Inc., December 15, 1970

SECTION 1. SCOPE

These specifications cover the design, manufacture and use of Open Web Steel Joists, **J**- and **H**-Series.

SECTION 2. DEFINITION

The term "Open Web Steel Joists **J**- and **H**-Series", as used herein, refers to open web parallel chord load-carrying members suitable for the direct support of floors and roof decks in buildings, utilizing hot-rolled or cold-formed steel, including cold-formed steel whose yield strength* has been attained by cold working. They are designed in accordance with these specifications to develop the resisting moments and maximum end reactions shown in the Standard Load Tables for Open Web Steel Joists, **J**- or **H**-Series attached hereto.

The design of **J**-Series joists shall be based on a yield strength of 36,000 psi and steel used for **J**-Series joists shall have a minimum yield strength of 36,000 psi in the hot-rolled condition prior to forming or fabrication.

The design of chord sections for **H**-Series joists shall be based on a yield strength of 50,000 psi. The design of web sections for **H**-Series joists shall be based on a yield strength of either 36,000 psi or 50,000 psi. Steel used for **H**-Series joists chord or web sections shall have a minimum yield strength determined in accordance with one of the procedures specified in Section 3.2, which is equal to the yield strength assumed in the design.

*The term "yield strength" as used herein shall designate the yield level of a material as determined by the applicable method outlined in paragraph 13 – "Yield Strength", or paragraph 12 – "Yield Point", of ASTM Standard A370, "Mechanical Testing of Steel Products", or as specified in Section 3.2 of this Specification.

SECTION 3. MATERIALS

3.1 STEEL

The steel used in the manufacture of chord and web sections shall conform to one of the following ASTM Specifications of latest adoption:

(a) Structural Steel, ASTM A36
(b) High-Strength Low-Alloy Structural Steel, ASTM A242
(c) High-Strength Low-Alloy Structural Manganese Vanadium Steel, ASTM A441
(d) Hot Rolled Carbon Steel Sheets and Strip, Structural Quality, ASTM A570.
(e) High-Strength Low-Alloy Columbium-Vanadium Steel of Structural Quality, ASTM A572 Grades 42, 45 and 50.
(f) High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield to 4" thick, ASTM A588.
(g) Hot Rolled or Cold Rolled Sheet, High-Strength Low-Alloy, with Improved Corrosion Resistance, ASTM A606
(h) Steel, Cold Rolled Sheet, Carbon Structural, ASTM A611, Type 2.

or shall be of suitable quality ordered or produced to other than the listed specifications, provided that such material in the state used for final assembly and fabrication is weldable and is proved by tests performed by the producer or fabricator to have the properties specified in Section 3.2.

3.2 MECHANICAL PROPERTIES

The yield strength used as a basis for the design stresses prescribed in Section 4 shall be either 36,000 psi or 50,000 psi. Evidence that the steel furnished meets or exceeds the design yield strength shall be provided in the form of witnessed or certified test reports.

For material used without consideration of increase in yield strength resulting from cold forming, the specimens

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If no drawings are available it is still possible to establish the approximate capacity of the member by field measuring the chord and web member sizes as well as the overall configuration of joist. This information can then be used to analyze the structure as a simple truss.

Critical assumptions that must be made with this approach include; the yield strength of the members, and if the existing panel point welds are capable of developing the full capacity of the connected component members.

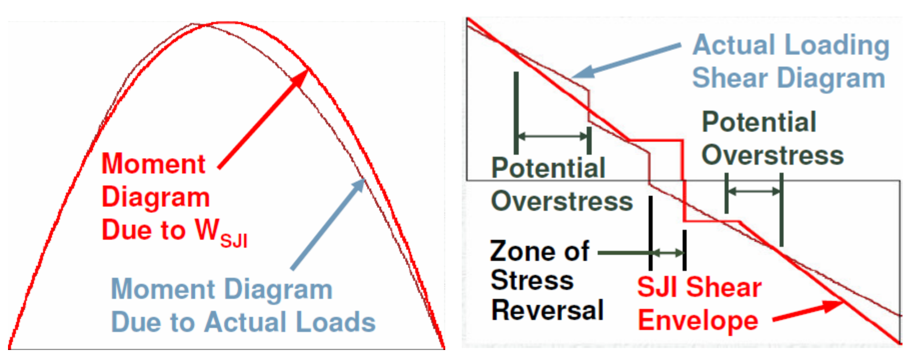
An alternate method to the above approach includes filling out the Joist Information Form located on the SJI website. SJI has indicated that they have been very successful in identifying the series and designation for many older joists with this resource.

Joist Investigation Form: http://www.steeljoist.org/investigation

- Engineers, Architects, Specifying Professionals, Contractors, and others trying to identify older joists found in the field can now fill out the form below, or they can use this downloadable form to provide the necessary information to the SJI office. Please fill out as much information as possible. This will help the SJI office in making a proper match of your joist information to those in our extensive historical files.
- When filling in the form regarding the joist chord and web member properties, it is recommended that the field measurements be taken with a micrometer rather than a tape measure, since chord thicknesses can vary by as little as 1/64 inch and web diameters can vary by 1/32 inch.
- Sending pictures or sketches of the joist profiles is also recommended when the member cross-sections seem to be of a proprietary nature. When you submit the form below and want to submit photographs or sketches to go along with it, please email them to sji@steeljoist.org

The next step in the evaluation process is to determine all of the existing loads on the joist system. The existing and new loading criteria are then used to establish the shear and moment envelope of the individual joist. This information is then used to compare to the allowable shear and moment envelope based on either the historical data provided by SJI or an independent analysis of the member as a simple truss.

If the SJI historical data is used for comparison to the actual loading on joists that where not fabricated with a uniform shear and moment capacity over the entire span length (i.e. not KCS joists) then in addition to confirming that the applied shear and moment do not exceed the joist capacity it is also necessary to compare the location of the maximum imposed moment to the midspan of the joists.

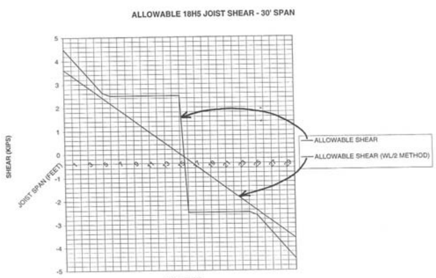


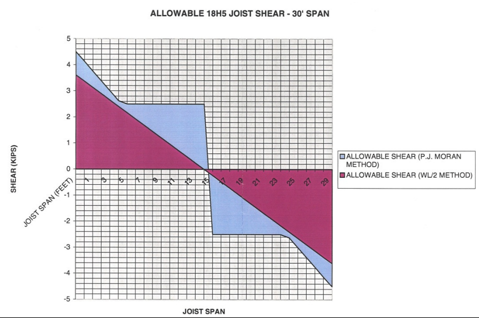
Source: Canam

Typically if the location of the maximum moment is less than or equal to one foot from the midspan and the maximum applied moment is less than the joist moment capacity, the joist is capable of safely supporting the imposed loads.

However, if the location of the maximum moment is greater than one foot from the midspan, the capacity of the joist may not be sufficient even if the applied moment is less than the specified capacity. This later situation can occur for two reasons. First the moment capacity envelope of the joist may actually be less in regions of the span other than plus or minus one foot from the midspan.

Secondly, a shift in the moment envelope from that normally associated with a uniformly loaded simple span (and the prerequisite shear envelope) may result in stress reversals in the web members (i.e. from tension to compression) that the original member was not designed or manufactured for. A similar, although typically more advantageous, condition also can occur with J or H-Series joists because of variations in the uniform shear capacity of these same members. (**Source of Images:** SJI)





In situations in which it has been confirmed that the existing joists do not have sufficient capacity to support the new loads there are three methods that can be used to rectify the condition:

1. **Load Redistribution.**
2. **Adding new joists or beams.**
3. **Reinforcing existing joists.**

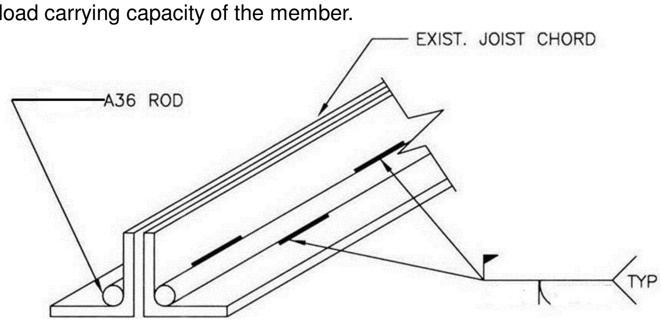
Source: www. thesanctuaryupc.com

Load redistribution involves the installation of a sufficiently stiff member perpendicular to the span of the joist as required to distribute the applied load to enough adjacent joists such that no one joist is overstressed as a result of the new loading.

Adding new joists or beams typically involves the installation of a new framing member parallel to the joist span such that all or most of the new applied load is supported by the new framing. New self supporting beams can also be installed perpendicular to the joist span as required to reduce the original span length of the member.

Finally, new independent, self supporting beam and column frames can also be installed to circumvent the imposition of any new loads on the existing joist framing system.

Reinforcing involves the installation of supplemental material to the original joist as required to increase the load carrying capacity of the member.



EXAMPLE OF BOTTOM CHORD REINFORCEMENT

The key to the successful use of load redistribution involves the installation of a structural member that can adequately and predictably distribute the applied load to enough adjacent joists to justify the safe support of the load. A method of calculating the relative stiffness of a distribution member is available in the reference material used for the development of this presentation.

In general, if the spacing of the joists is less than approximately 78% of the calculated stiffness of the distribution member and the length of the distribution member is less than the inverse of the calculated stiffness, then the distribution member may be considered as rigid enough to statically calculate the load reactions to the affected joists.

The relative stiffnesses of the joists and the distribution beam is defined by the characteristic parameter beta as defined in Equation 5.5.1.

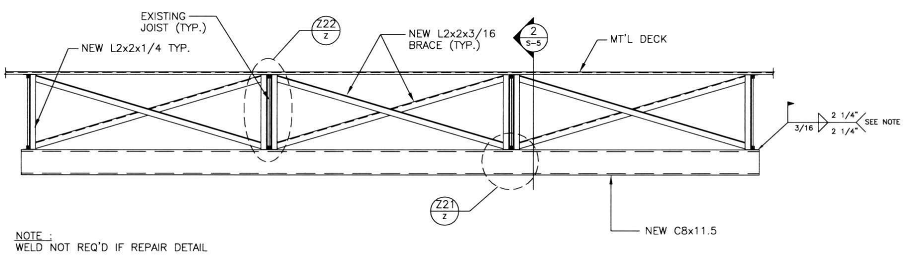
$$\beta = \sqrt[4]{(K/S)/(4EI)}$$

Where: K = The stiffness of the joist, kips/inch
S = The spacing of the joists
E = The modulus of elasticity for the beam
I = The moment of inertia of the beam

If S is less than $\pi/4\beta$ the beam on elastic support calculations are applicable. If the spacing limit is not exceeded and the length of the beam is less than $1/\beta$, the beam may be considered to be rigid with respect to the supporting joists and the reactions to the joists may be determined by static equilibrium.

For load redistribution solutions it is my preference to use trussed distribution members rather than individual beams to assure the adequate transfer of the applied load.

By trussed the author means continuous members located perpendicular to both the existing joist bottom and top chords in conjunction with diagonal web members connected to the continuous members at the intersection of the joist chords. The resulting configuration looks like a truss and provides greater stiffness than an individual beam connected to either the joist bottom or top chords. The author also recommends that no more than five joists be engaged by a distribution member. In addition, the use of pipes for the continuous distribution truss chord members can also be advantageous as this type of section fits neatly through the V shaped panel point openings created at the intersection of the existing chords and web members. Load redistribution solutions may be difficult to install depending on accessibility and the presence of existing MEP systems, ceilings or other appurtenances.

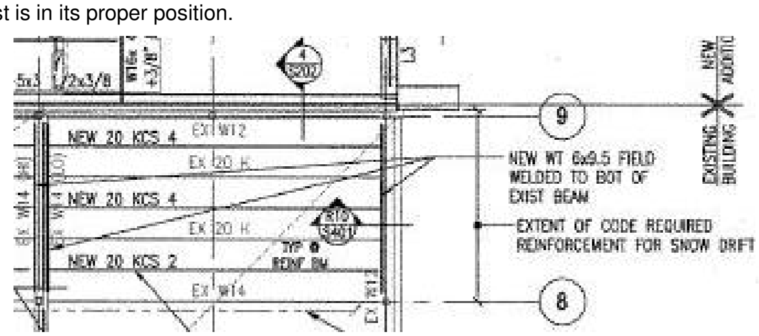


Trussed Distribution Detail

As indicated above, adding new joists or beams to an existing system can also be used to provide solutions to new loads on a joist structure. When new members are added parallel to the existing joists the new framing can be used to either reduce the tributary area of the existing joists or provide direct support of the new loads such that there is no impact on the existing joist framing.

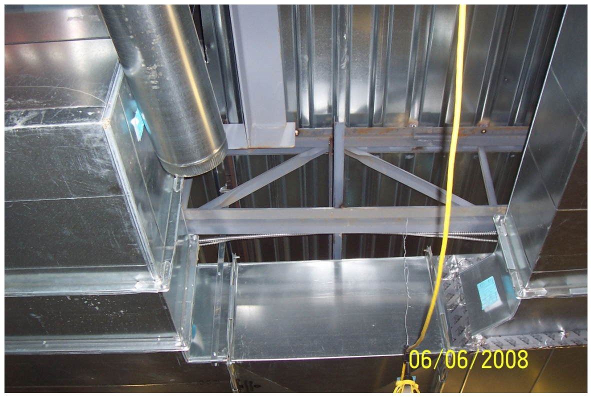
Methods used to install new parallel framing often involve the need to manufacture, ship and erect the new members using field splices. However, it is possible to install new full length manufactured joists via the use of loose end bearing assemblies.

In this later scenario the joists are first erected on a diagonal to allow the top chord to be lifted above the bearing elevation. The joist is then rotated into an orthogonal position with the lower portion of the bearing assembly then dropped and welded into place. Typically in this situation, a shallower bearing seat is also provided for ease of installation and then shimmed once the new joist is in its proper position.



When new beams or other similar members are added perpendicular to the joist span the new framing serves to reduce the span of the existing members thereby increasing the load carrying capacity of the joists. However, in this scenario it is still necessary to analyze the existing joists to assure that no load reversals have occurred in tension only web members and that the actual applied moment falls within the remaining existing moment capacity envelope of the joist.

As with load redistribution solutions, both of the above new framing approaches may be difficult to install depending on accessibility and the presence of existing MEP systems, ceilings or other appurtenances.



Typical Ceiling Congestion

New framing that involves the installation of independent stand alone beam and column frames is intended to provide direct support of the new loads such that there is no impact on the existing joist framing. This type of new framing can involve beams (located either beneath or above the impacted existing framing) supported by new columns and foundations or beams that frame between existing columns.

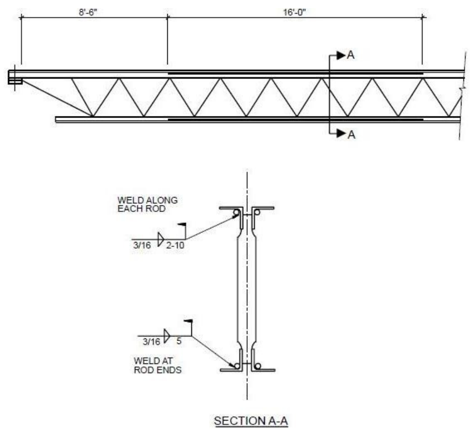
This type of solution can also involve new beam frames supported from posts located directly above existing beams or columns. The above solutions are typically less susceptible to the presence of existing MEP systems, ceilings or other appurtenances as the other new beam or joist framing solutions. (**Source of Image:** Kiwi Steel Corp.)

Procedures for reinforcing joists are expertly described in the SJI Technical Digest No. #12 and involve two basic approaches:

1. Ignore the strength of the existing member and simply design the new reinforcement to carry all of the applied load, or…

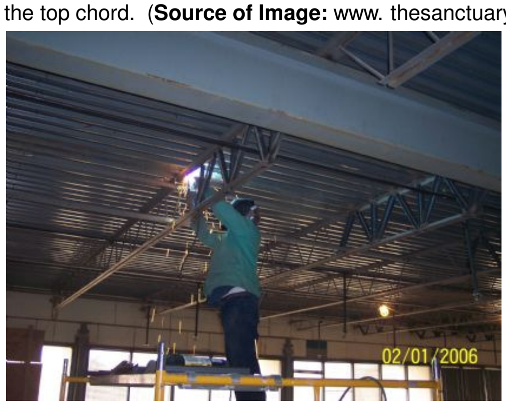
2. Make use of the strength of the existing members when designing the reinforcing.

Both of the recommended approaches typically involve significantly more labor costs than material costs because of the expense associated with field welding. (**Source of Image:** Vulcraft)



I prefer to avoid the use of field reinforcement for the following reasons. A manufactured open web steel joist is basically a pre-engineered product, however, when an engineer involved with the modification of an existing joist specifies new field installed reinforcement, that same engineer assumes the responsibility for the overall adequacy of the joist. This liability extends to not only the reinforcing modifications but also inherently to any pre-existing, unknown conditions or deficiencies in the joist. In addition, field welding associated with the installation of reinforcement also poses concerns for the design engineer.

Problems associated with field welding are also discussed in Technical Digest No. #12 and include; temporary localized loss of the material strength of the existing steel due to heat generated by the weld, induced eccentricities, inadequate load path mechanisms, and lack of access particularly at the top chord. (**Source of Image:** www. thesanctuaryupc.com)



The only exceptions that the author makes relative to reinforcing joists in his own practice includes: the installation of supplemental web members as needed to transfer concentrated loads greater than 150 pounds on chords that are located greater than 6 inches from a panel point to the closest adjacent panel point (see Diagram A),

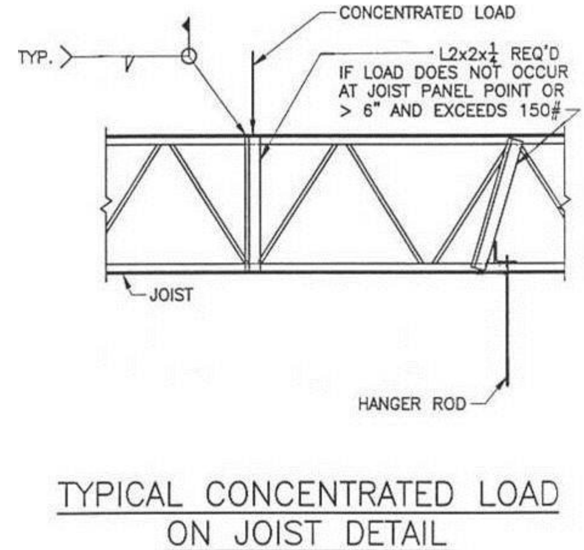
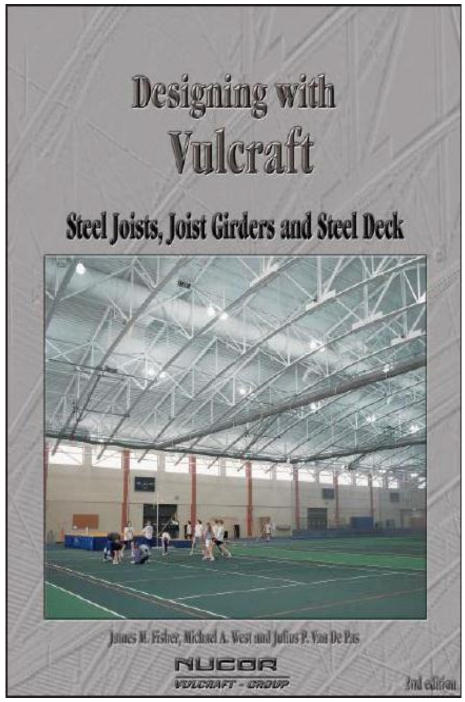


Diagram A

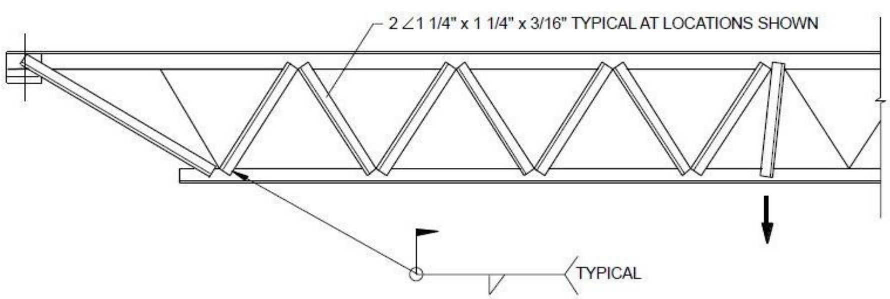
…and reinforcement designed by the original manufacturer's engineer. should also be noted that a new proprietary "weldless" joist reinforcement system is now manufactured by Lindapter.

The analysis of existing open web steel joists can be a challenging undertaking and often involves a considerable amount of detective work. Unfortunately, there is typically little or no documentation available concerning the capacity of a specific existing joist under investigation. However, it is hoped that the reference information provided in this presentation will assist in increasing the likelihood that the capacity of a joist can be determined using the historical data that is available through the SJI. Additional resources for the analysis and modification of existing joists are also provided by Vulcraft.



Typically the investigation of an existing joist results in the need to modify the existing structural system to provide for the support of new imposed loads.

At this juncture, the engineer must then determine if he or she is more comfortable with either assuming the responsibility and liability for modifying a pre-engineered product or deciding if a possibly less risky option such as load redistribution or adding new joist or beam framing are more appropriate.



Joist Diagonal Reinforcement

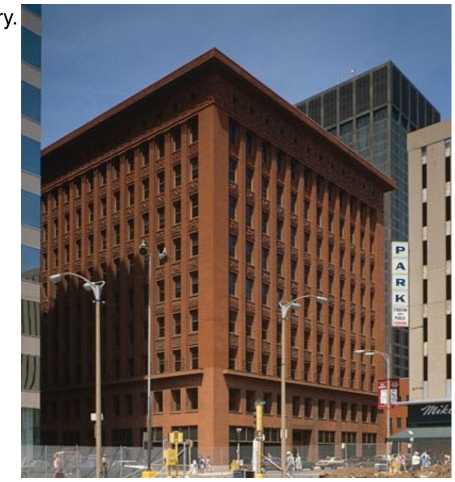
Source: Vulcraft

Additional Antiquated Systems

Masonry:

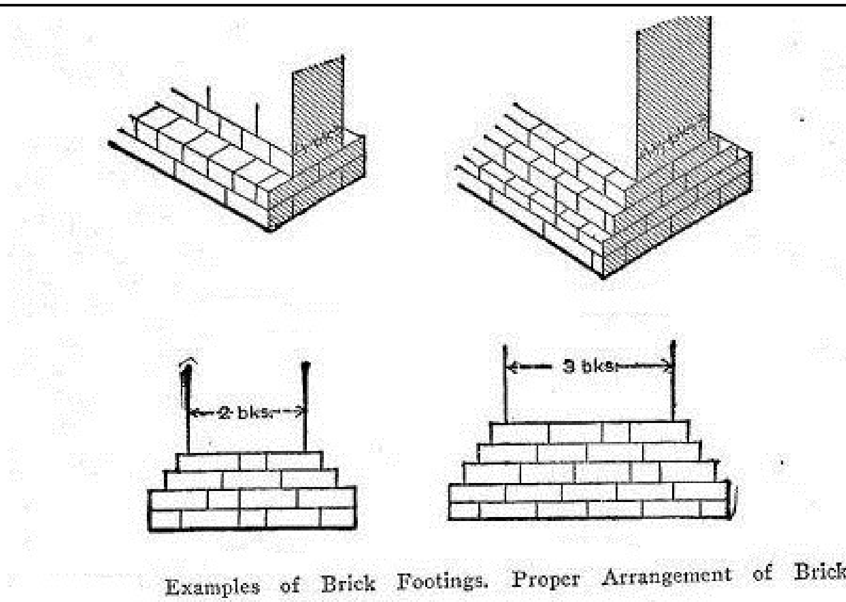
Masonry bearing walls were rarely if ever designed for the actual loading conditions. However, an analysis of a typical 8 inch double wythe masonry brick wall for a typical 3 to 5 story building indicates that the compressive stresses are well below the allowable for common masonry brick of the 20th Century.

Example of Steel Framed Masonry Building



Wainwright Building

Source: Historic American Building Survey



Masonry Brick Foundations

Source: Building Construction and Superintendence

Building codes in New York City first addressed masonry walls in 1830. The code provisions for masonry brick became more complicated with each revision until by 1892 the portion of the code dealing with masonry was the most complex part of the code. The NYC code, as did many other codes from different major cities, specified the minimum wall thickness for varying heights of buildings. The 1892 NYC code generally called for an increase of 4 inches (i.e. one wythe of brick) in wall thickness for each 15 feet down from the top of the building.

The minimum thickness for "curtain" masonry brick walls was generally 4 inches less than that required for a loadbearing walls at the same height of the building. As it is sometimes difficult to ascertain the thickness of brick masonry walls in existing buildings a table listing the various minimum wall thicknesses has been provide below for a number of major cities from the 1920's.

BUILDING CODE

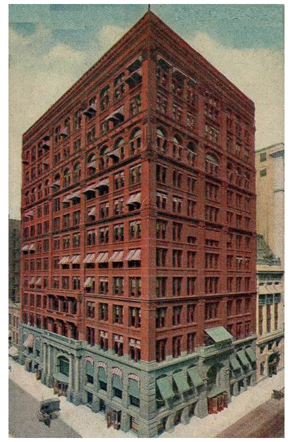
OF THE

CITY OF NEW YORK.

Minimum Building Code Thickness of Brick Masonry Walls - Inches									
Total Stories	City	Floor							
		1st	2nd	3rd	4th	5th	6th	7th	8th
2	Boston	12	12						
	New York	12	12						
	Chicago	12	12						
	Philadelphia	13	13						
	Denver	12	12						
	San Francisco	17	13						
3	Boston	12	12	12					
	New York	16	16	12					
	Chicago	16	12	12					
	Philadelphia	18	13	13					
	Denver	16	12	12					
	San Francisco	17	17	13					
4	Boston	16	12	12	12				
	New York	16	16	16	12				
	Chicago	20	16	16	12				
	Philadelphia	18	18	13	13				
	Denver	16	16	12	12				
	San Francisco	17	17	17	13				
5	Boston	16	16	12	12	12			
	New York	20	16	16	16	16			
	Chicago	20	20	16	16	16			
	Philadelphia	22	18	18	13	13			
	Denver	20	20	16	16	12			
	San Francisco	21	17	17	17	13			
6	Boston	16	16	16	12	12	12		
	New York	24	20	20	16	16	16		
	Chicago	20	20	20	16	16	16		
	Philadelphia	22	22	18	18	13	13		
	Denver	20	20	20	16	16	12		
	San Francisco	21	21	17	17	17	13		
7	Boston	20	16	16	16	12	12	12	
	New York	28	24	24	20	20	16	16	
	Chicago	20	20	20	20	16	16	16	
	Philadelphia	26	22	22	18	18	13	13	
	Denver	24	20	20	20	16	16	12	
8	Boston	20	20	16	16	16	12	12	12
	New York	32	28	24	24	20	20	16	16
	Chicago	24	24	20	20	20	16	16	16
	Philadelphia	26	26	22	22	18	18	13	13
	Denver	24	24	20	20	20	16	16	12

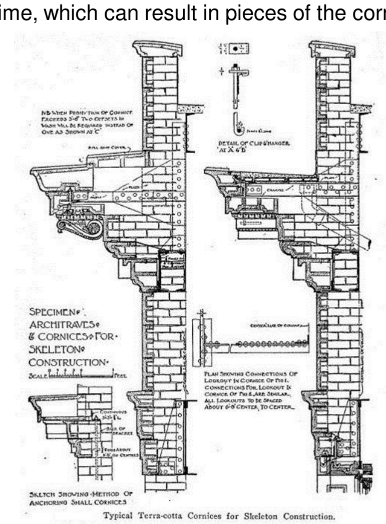
The use of loadbearing brick masonry walls were eventually replaced by cage and skeleton wrought-iron and steel frame construction (using cast-iron columns). **Cage** construction involved the use of brick façade walls that were as thick as that used for loadbearing construction, the only difference was that the frame and supporting columns (including those that would eventually be embedded in the brick masonry façade wall) were first erected ahead of the masonry. **Skeleton** framing, although partially embedded in the exterior masonry walls, was only clad with what amounted to a brick curtain wall. All three of these forms of construction co-existed between 1880 and 1900. (**Sources of Images:** www.skyscraperpage.com and www.kootation.com)

Skeleton Construction
Phelan Building San Francisco



Cage Framing
Home Insurance Building Chicago

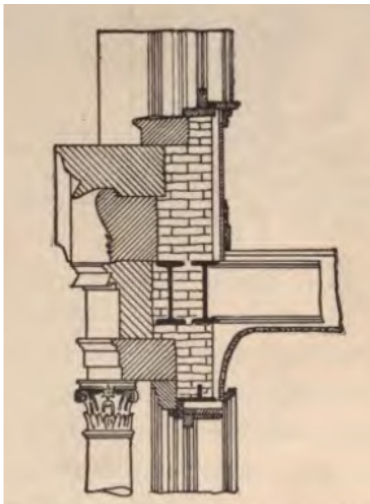
Another area that structural engineers often get involved with in older masonry structures is façade restoration. Here is an illustration of a typical Terra Cotta cornice in which the individual pieces of hollow terra cotta are supported by steel outriggers. It is common for the supporting steel to deteriorate over time, which can result in pieces of the cornice falling to the sidewalk below.



Terra Cotta Cornice

Source: Building Construction and Superintendence

Here is an example of another common condition that can be encountered in antiquated steel framed structures with masonry cladding – a double spandrel beam condition. Typically the outboard beam was designed to support the masonry loads only and the inboard beam was designed to support the floor loading, and a portion of the exterior wall load as well.



Double Spandrel Condition

Source: Freitag

Floor Framing:

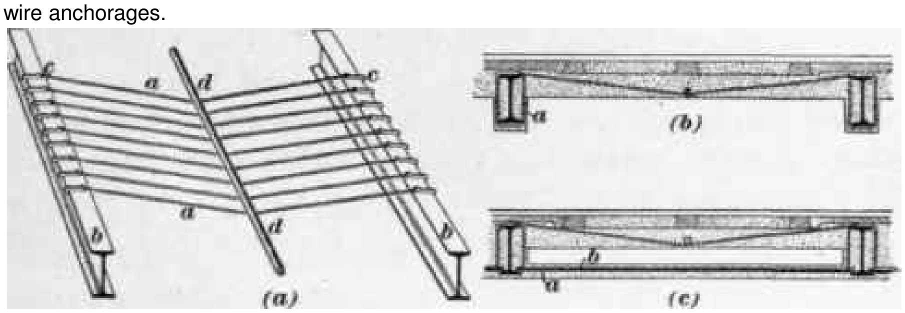
Draped mesh slabs became popular in the 1920's. Draped mesh construction is a type of reinforced slab framing that involves the use of wires that drape between the tops of adjacent beams. The types of mesh used included triangular wire mesh, ordinary wire mesh, expanded metal sheets and plain round and square and twisted square rods.

Draped Mesh Slab

Draped Mesh Slab

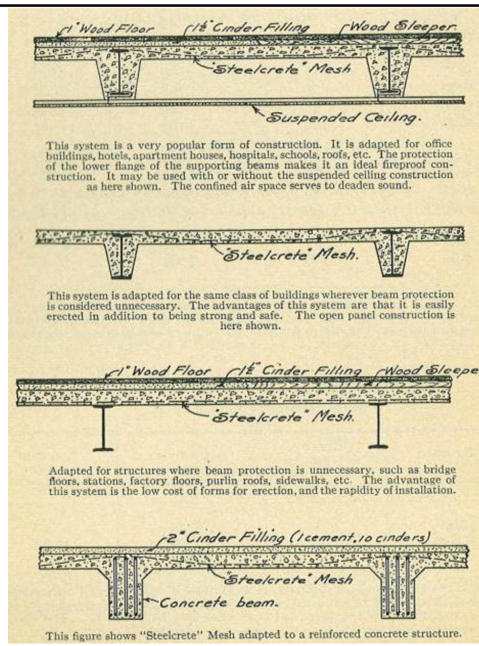
The use of wire mesh was actually preceded by expanded metal sheets. Welding of wires together to form the mesh did not begin until the 1930's. Prior to that the wires were attached at the intersection points by either staples, washers or by wrapping the transverse wires around the longitudinal wires.

In a draped mesh slab the concrete serves only as the wear surface and as the mechanism by which the imposed loads are transmitted to the mesh. The mesh alone is what physically spans between the beams via catenary action. Because the concrete is not structurally stressed in this type of system the composition and quality of the concrete is not as important as in a true flexural slab. As a result it was common to use cinder concrete with compressive strengths under 1,000 psi. The use of cinder concrete, however, due to the acidic nature of the clinker (coal cinder) used as the aggregate resulted in the corrosion of the embedded iron beams and reinforcing mesh. Catenary systems are also vulnerable to collapse as a result of failure of the wire anchorages.



Metropolitan System

Source: A Treatise on Architecture and Building Construction, Volume 2



This system is a very popular form of construction. It is adapted for office buildings, hotels, apartment houses, hospitals, schools, roofs, etc. The protection of the lower flange of the supporting beams makes it an ideal fireproof construction. It may be used with or without the suspended ceiling construction as here shown. The confined air space serves to deaden sound.

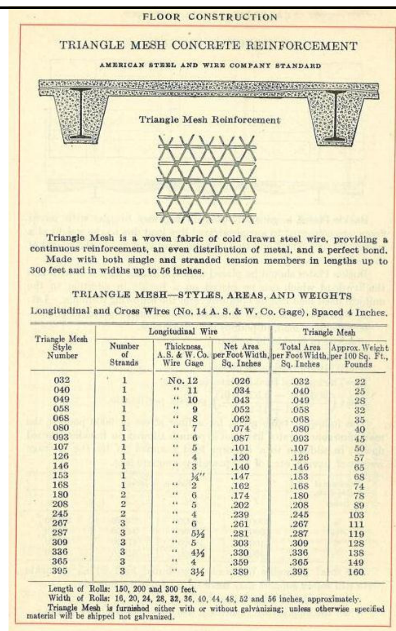
This system is adapted for the same class of buildings wherever beam protection is considered unnecessary. The advantages of this system are that it is easily erected in addition to being strong and safe. The open panel construction is here shown.

Adapted for structures where beam protection is unnecessary, such as bridge floors, stations, factory floors, purlin roofs, sidewalks, etc. The advantage of this system is the low cost of forms for erection, and the rapidity of installation.

This figure shows "Steelcrete" Mesh adapted to a reinforced concrete structure.

Steelcrete Draped Mesh Floor Systems

Source: Building Construction and Superintendence


FLOOR CONSTRUCTION

TRIANGLE MESH CONCRETE REINFORCEMENT

AMERICAN STEEL AND WIRE COMPANY STANDARD

Triangle Mesh is a woven fabric of cold drawn steel wire, providing a continuous reinforcement, an even distribution of metal, and a perfect bond.

Made with both single and stranded tension members in lengths up to 300 feet and in widths up to 56 inches.

TRIANGLE MESH—STYLES, AREAS, AND WEIGHTS

Longitudinal and Cross Wires (No. 14 A. S. & W. Co. Gage), Spaced 4 Inches.

Triangle Mesh Style Number	Longitudinal Wire			Triangle Mesh	
	Number of Strands	Thickness, A.S. & W. Co. Wire Gage	Net Area per Foot Width, Sq. Inches	Total Area per Foot Width, Sq. Inches	Approx. Weight per 100 Sq. Ft., Pounds
032	1	No. 12	.026	.032	22
040	1	" 11	.034	.040	25
049	1	" 10	.043	.049	28
058	1	" 9	.052	.058	32
068	1	" 8	.062	.068	35
080	1	" 7	.074	.080	40
093	1	" 6	.087	.093	45
107	1	" 5	.101	.107	50
126	1	" 4	.120	.126	57
146	1	" 3	.140	.146	65
153	1	¼"	.147	.153	68
168	1	" 2	.162	.168	74
180	2	" 6	.174	.180	78
208	2	" 5	.202	.208	89
245	2	" 4	.239	.245	103
267	3	" 6	.261	.267	111
287	3	" 5½	.281	.287	119
309	3	" 5	303	.309	128
336	3	" 4½	.330	.336	138
365	3	" 4	.359	.365	149
395	3	" 3½	.389	.395	160

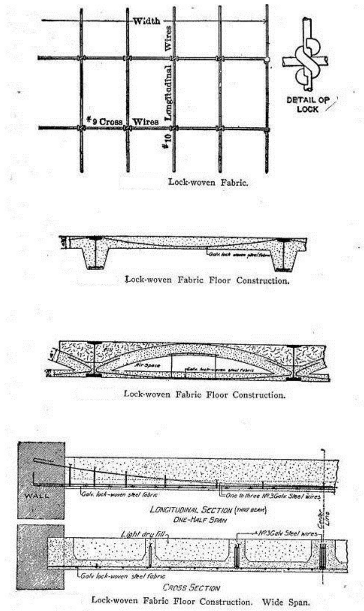
Length of Rolls: 150, 200 and 300 feet.

Width of Rolls: 16, 20, 24, 28, 32, 36, 40, 44, 48, 52 and 56 inches, approximately.

Triangle Mesh is furnished either with or without galvanizing; unless otherwise specified material will be shipped not galvanized.

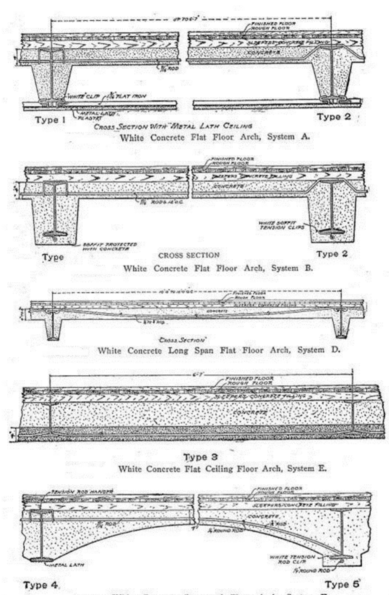
Triangle Draped Mesh Floor Reinforcement

Source: Building Construction and Superintendence



Lock-Woven Draped Mesh Floor Systems

Source: Building Construction and Superintendence

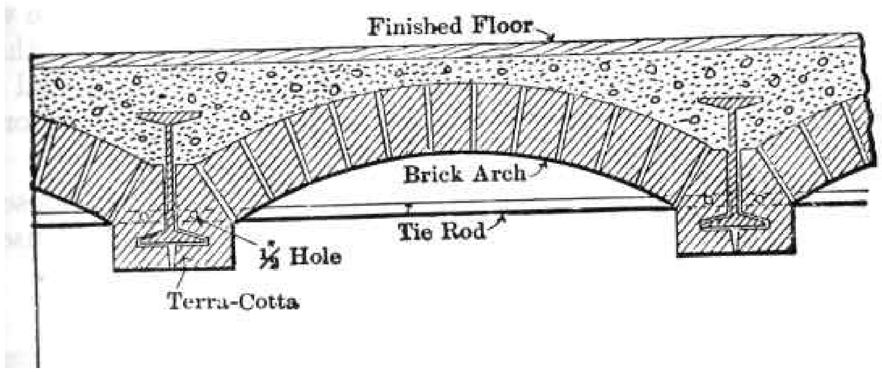


White Concrete Draped Mesh & Arched Floor Systems

Source: Building Construction and Superintendence

Brick arch floor construction consisted of a single arch of unmortared brick (typically only one wythe or 4 inches thick) capable of spanning 4 to 8 feet with a center rise of approximately 1/8 of the span. The spring line of the arch was constructed on top of the bottom flange of the supporting beams. The space above the arch was filled in with concrete which sometimes had wood nailer strips embedded in the top of the slab.

Tie rods were commonly placed about 1/3 of the height of the beam and were spaced from 4 to 6 feet on center. The entire system had to be built on formwork which supported the brick.



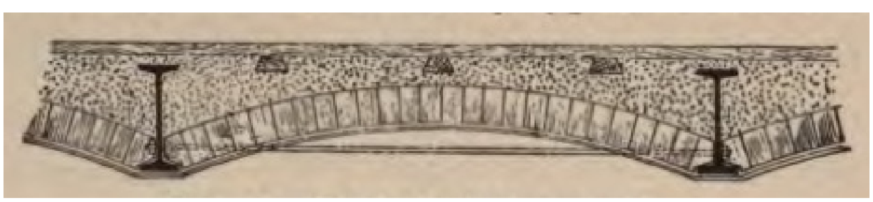
Brick Arch Floor

Source: Kidder-Parker

The thrust (T) from the arch in pounds per linear feet can be calculated as follows:

$$T = (1.5 \times W \times L^2)/R$$

Where: W = load on the arch in PSF
L = span length of the arch in feet
R = rise of the arch in inches.



Source: Architectural Engineering

Other antiquated floor systems include:

Fawcett System and Acme Floor-Arch – clay lateral cylindrical tile flat end construction arch.

Rapp Floor and McCabe Floor – gauge-steel inverted tees spaced at approximately 8 inches on center supporting a layer of brick and upper cinder concrete slab spanning 4 feet between supporting beams.

Roebling Floor Arch – arch of dense wire mesh supported on the top of the bottom flanges of the beams covered with concrete.

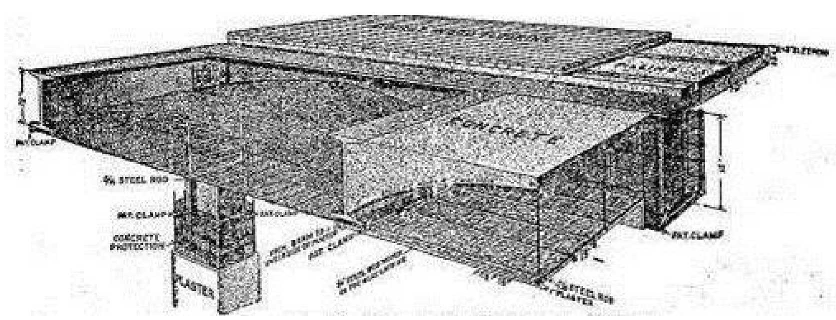
Manhattan System and Expanded Metal Company (EMC) Floor – flat and arched (for EMC) expanded metal mesh covered with concrete.

Multiplex Steel-Plate, Buckeye and Pencoyd Corrugated Floor – Riveted steel plates supporting a concrete slab.

Thompson Floor – Unreinforced concrete slab spanning approximately 3'-6" between beams connected with tie rods.

Roebling Flat Slab Floor and Columbian Floor System – reinforced concrete slab.

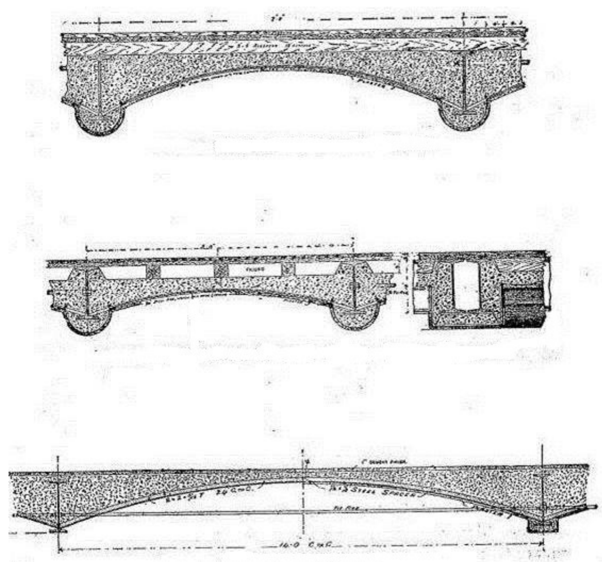
The Roebling arched floor, or System A, was an unreinforced arched concrete slab. The concrete was cast on top of stiffened arched wire cloth "centering" or permanent formwork. Additional wire lath was sometimes also suspended below the arched wire cloth to allow for a smooth plaster ceiling. This system was capable of 8-feet spans with a load carrying capacity of 200 PSF based on a safety factor of 4.



Roebling Arched Floor

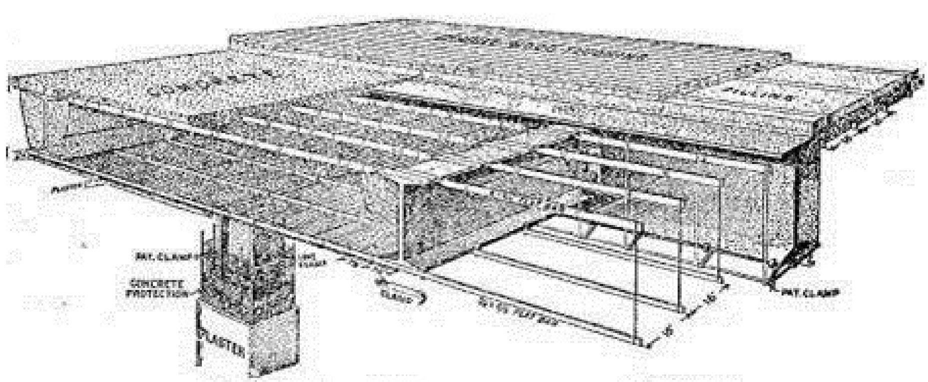
Source: Building Construction and Superintendence

Other variations of the arched Roebling floor system are shown on this slide.



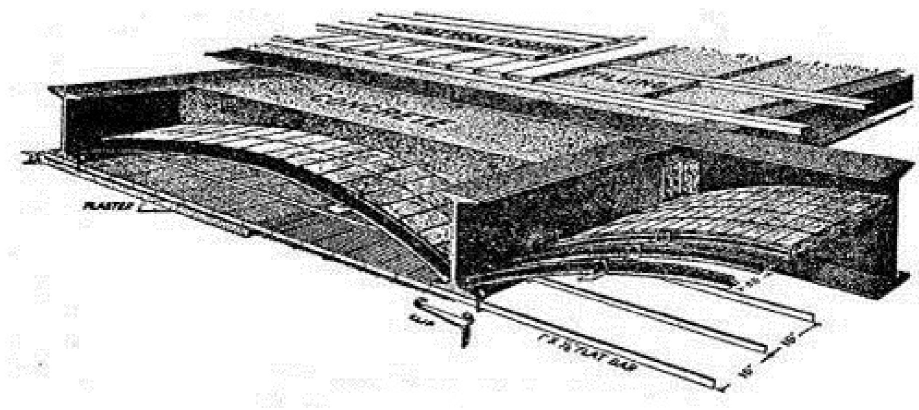
Source: Building Construction and Superintendence

The Roebling flat floor, or System B, was a true reinforced concrete slab. The reinforcing was provided by flat bars, which also served to support the stiffened wire cloth "centering" or permanent formwork. The bars were sometimes "trussed" or bent down into the bottom of the concrete at the midspan. This system was capable of spans of 10 and 12-feet.



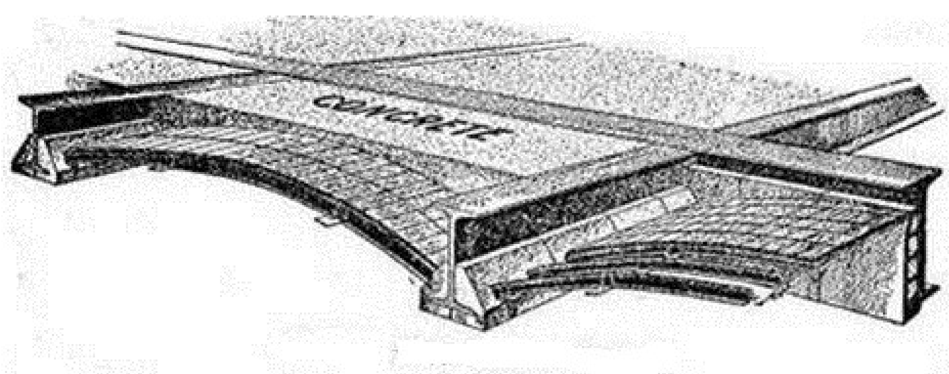
Roebling Flat Floor System

Source: Building Construction and Superintendence



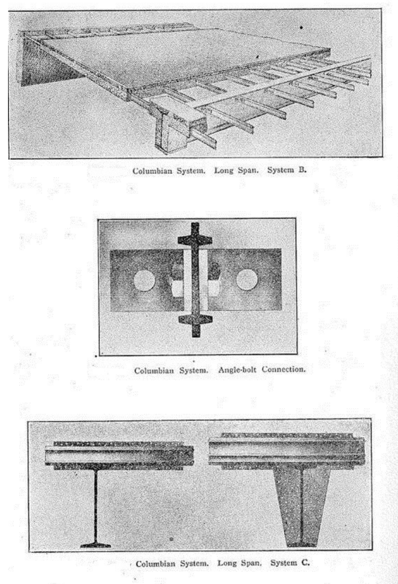
Rapp Floor System Type A

Source: Building Construction and Superintendence



Rapp Floor System Type C

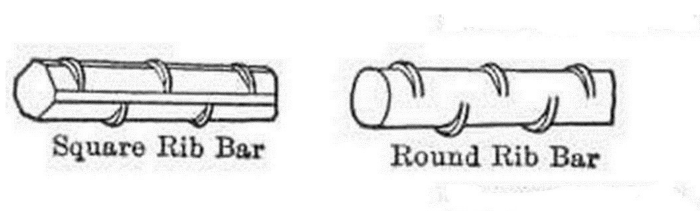
Source: Building Construction and Superintendence



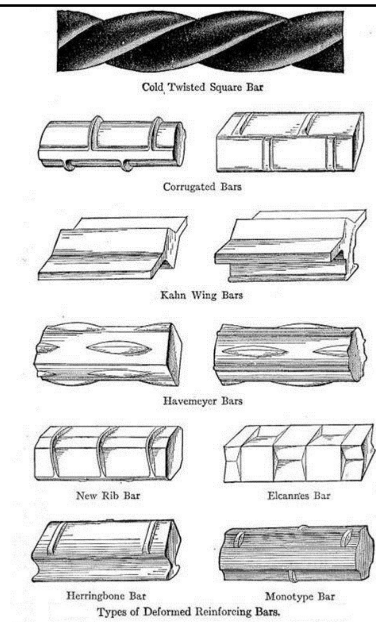
Columbian Floor System

Source: Building Construction and Superintendence

Plain round and square bars were typically used in reinforced concrete buildings built before 1920. Plain bars began to be phased out during the 1910's and early 1920's in favor of deformed bars. The two types of deformations used at that time included longitudinal and radial deformations. In addition, the Ransome bar included deformations induced by twisting square bars.

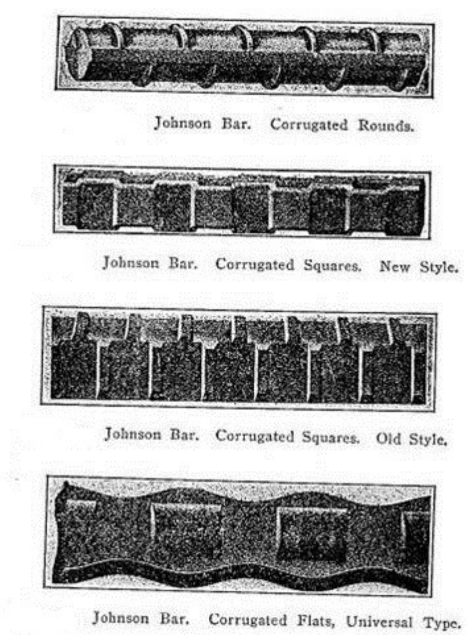


Source: Truscon Steel Co.



Additional Deformed Bars

Source: Pocket Companion



Johnson Deformed Bars

Source: Building Construction and Superintendence

Priddle Deformed Bar

Source: Building Construction and Superintendence

Other forms of longitudinally deformed bars included:

Thatcher bar which was a square bar with crossed shaped deformations on each face.

Lug bar which was a square bar with small round projections at the corners.

Inland bar which was a square bar with raised stars on each face.

Herringbone, **Monotype** and **Elcannes** bars which included complex cross sections similar to radial deformed bars but with longitudinal deformations.

Havemeyer bar which included round, square and flat cross sections with diamond plate type deformations.

Rib bar which included a hexagonal cross section with cup shaped deformations.

American bar with square and round cross sections and low circumferential depressions.

Scofield bar with an oval cross section and discontinuous circumference ribs.

Corrugated bar with flat, round and square cross sections with cup deformations.

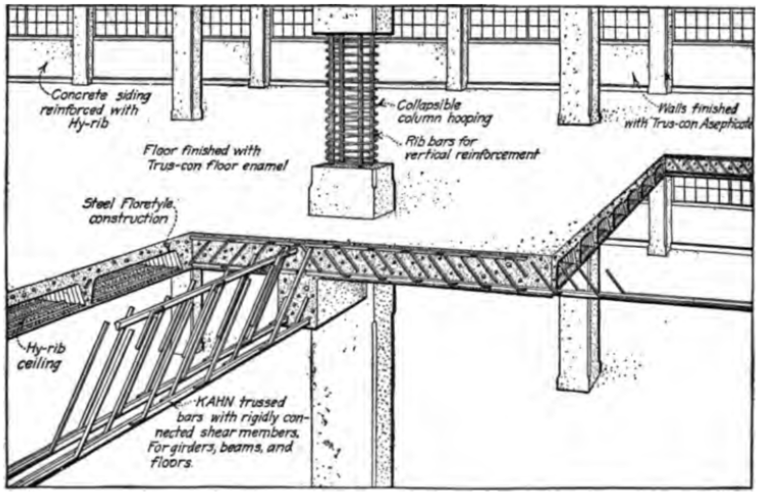
Slant bar with a flat cross section and low projecting diagonal ribs on the flat faces.

Cup bar with a round cross section and cup deformations.

Diamond bar with a round cross section and low circumferential ribs.

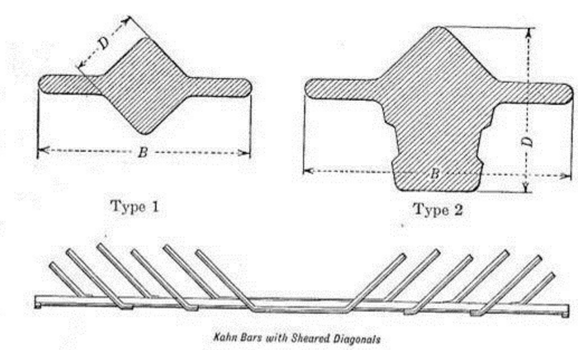
The modern designation of #3 to #8 round cup or diamond deformed bars were established in 1924.

Reinforcing for concrete beams was also available in prefabricated trussed bar units. A truss bar is essentially a top bar at the ends of a beam that is bent diagonally down to a bottom bar position at the midspan. Prefabricated assemblies included the **Kahn System**, the **Cumming System**, the **Corr System**, the **Hennebique System**, the **Pin-Connected System**, the **Luten Truss** and the **Xpantruss System**.



Kahn System

Source: Handbook of Building Construction



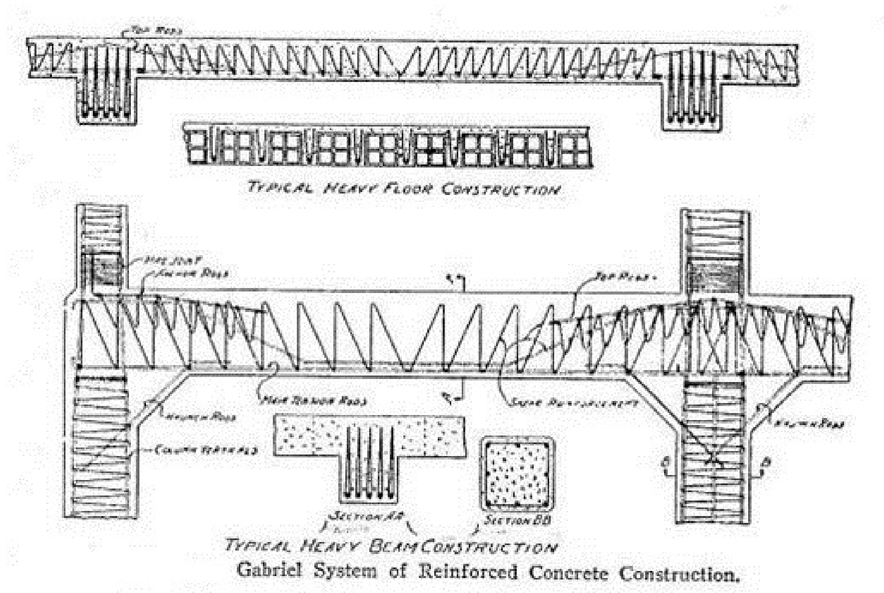
Sections of Kahn Trussed Bars.

Herringbone Bar.—The sizes and weights of the Herringbone bars are as follows:

Size, inches	1½	1¼	1	⅞	¾	⅝
Weight per foot, lb. . .	5.13	3.62	2.38	1.72	1.28	0.91

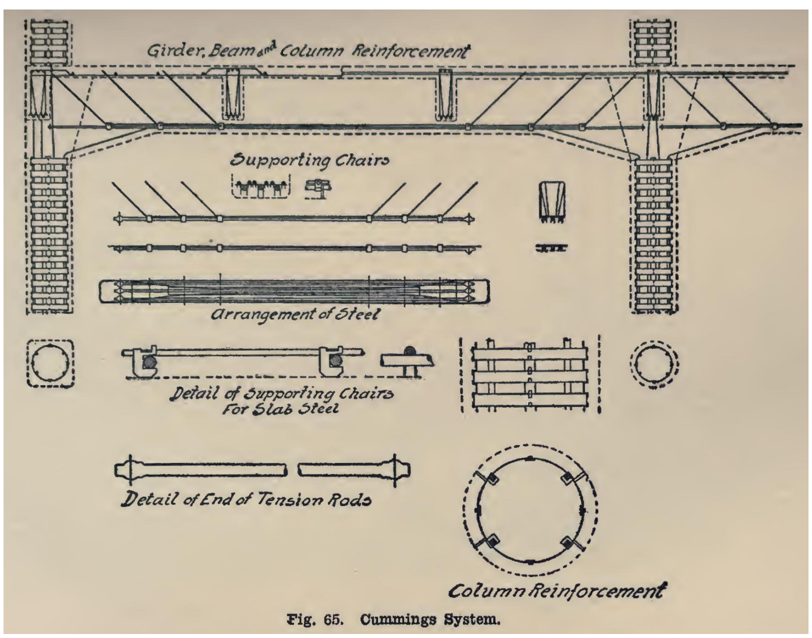
Kahn Herringbone Bar

Source: Truscon Steel Co.



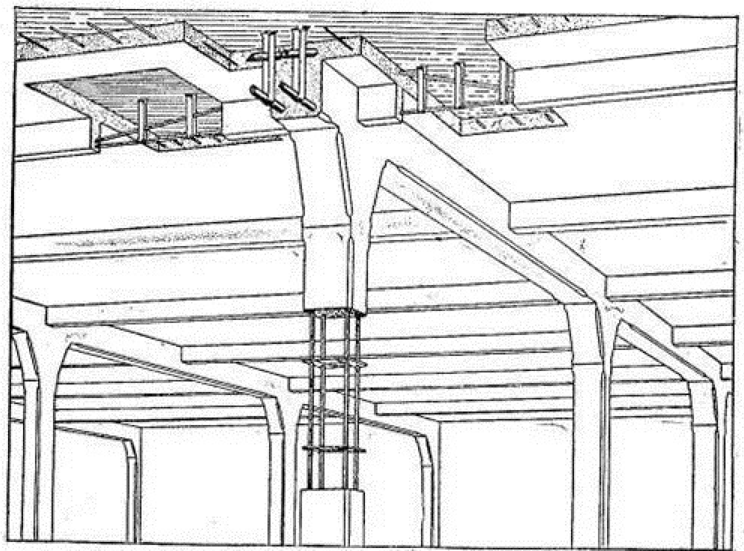
Gabriel Reinforced Concrete System

Source: Radford's Cyclopedia of Construction



Cummings Reinforced Concrete System

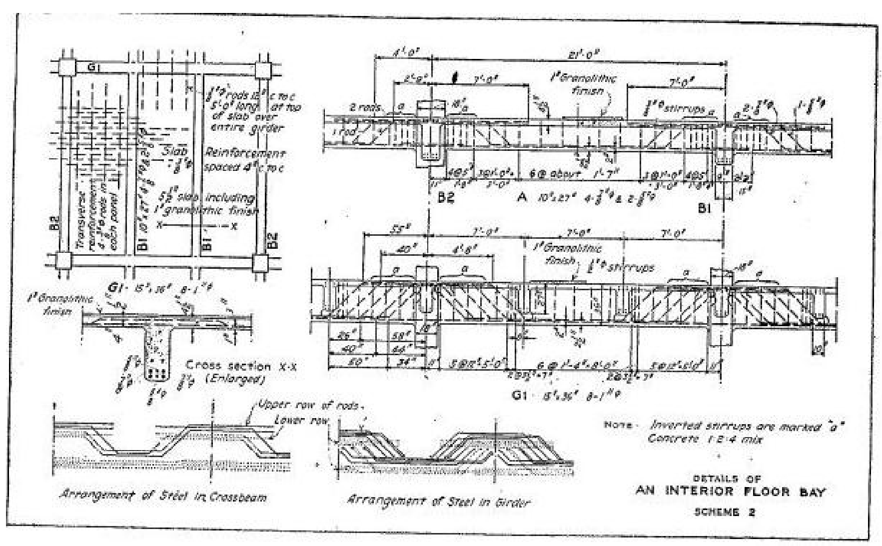
Source: Radford's Cyclopedia of Construction



Hennebique System of Reinforced Concrete Construction.

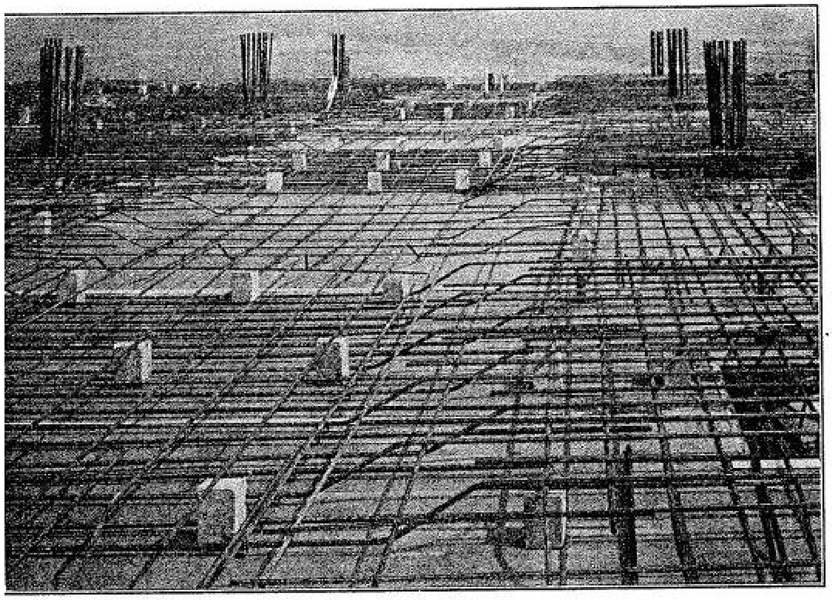
Hennebique Reinforced Concrete System

Source: Building Construction and Superintendence



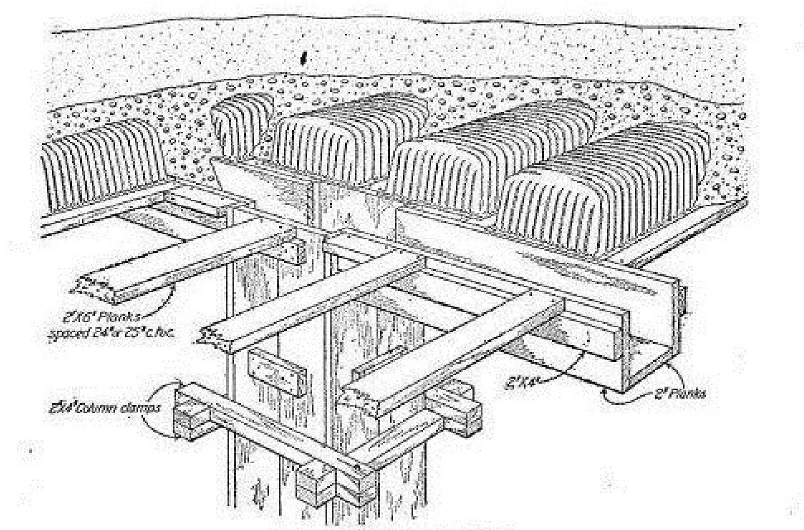
Trussed Top Bar Beam Reinforcing

Source: Concrete Engineers Handbook



Trussed Top Bar Slab Reinforcing

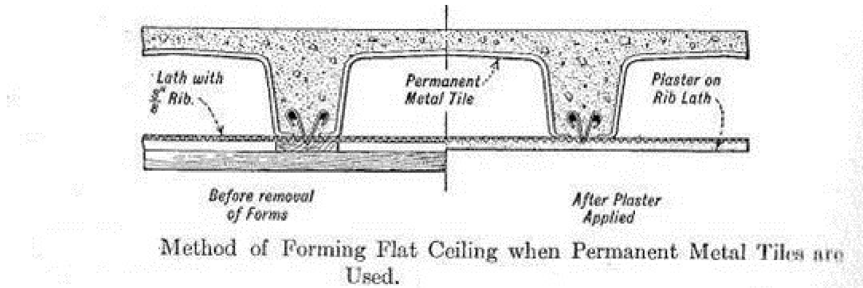
Source: Concrete Engineers Handbook



G. F. steel-tile.

GF Steel Tile System

Source: Concrete Engineers Handbook

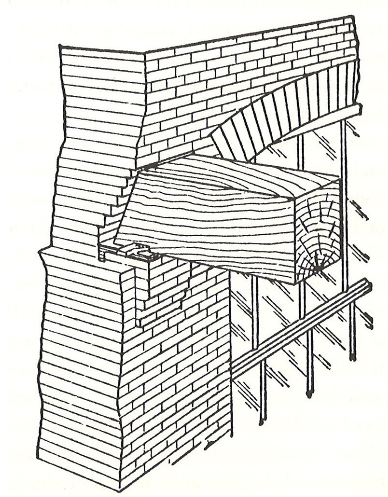


Metal Tile Floor and Ceiling System

Source: Building Construction and Superintendence

Wood:

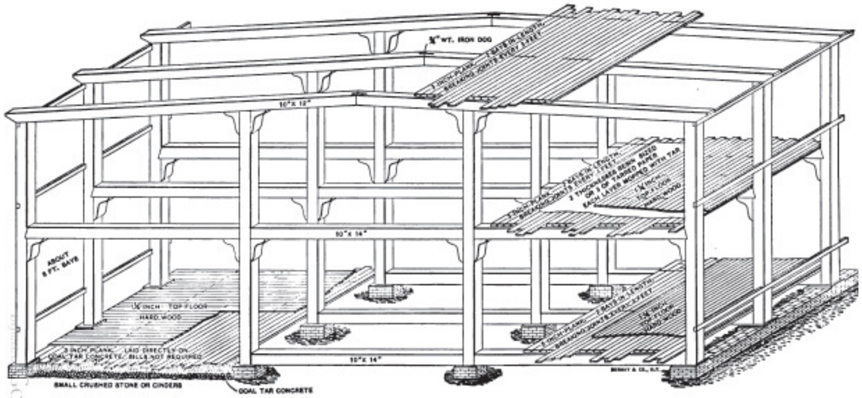
This side shows a fire cut end of a timber joist that was commonly used to allow for a wood beam that was on fire to rotate out of the beam pocket in a masonry wall without damaging the brick wall.



Fire Cut Joist End

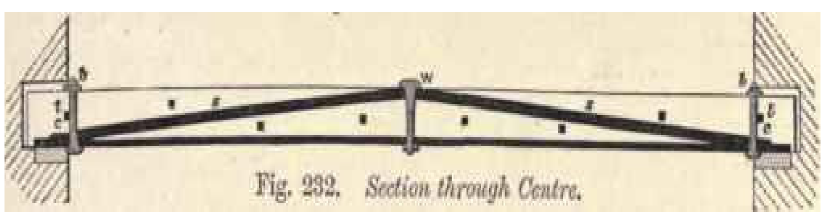
Source: Freitag

A system of timber construction that was used exclusively in industrial mill buildings in the late 1800's is illustrated on this slide and was considered to be "slow-burning" rather than fire resistance and therefore provided some measure of safety to the occupants.

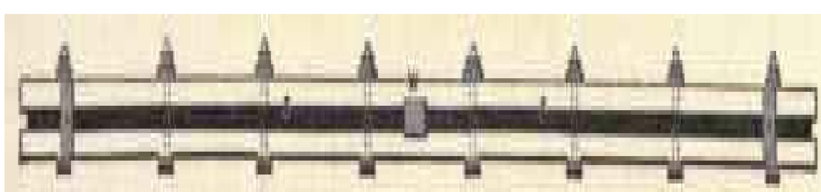


Source: Construction Equipment and Management

Another interesting form of wood construction that I have encountered that involves composite materials is a Trussed-Beam. This system involves positioning iron rods between two adjacent wood beams to form essentially an inverted king-post truss.



Elevation



Plan

Source: Notes on Building Construction

Trussed-Beam
USS Constitution Museum

Fire Resistance Ratings of Heavy Timber Construction:

- Fire resistance ratings and sprinkler requirements are listed in Table 601 of IBC 2006, which indicates that Heavy Timber (HT) or Type IV construction has advantages over other non-combustible types of construction. This is because HT construction has greater fire resistance than unprotected structural steel.

- IBC 2006 specifies minimum HT dimensions of 8 inches for columns per Section 602.4.1 and 6 inches (width) x 10 inches (depth) for floor framing per Section 602.4.2. In addition, floors must be constructed with splined or tongue-and-grove planks of not less than 3-inch thickness covered with 1-inch tongue-and-grove floor deck laid orthogonally or diagonally to the span of the plank. There are also prescriptive framing and connectivity requirements for HT construction specified in Section 2304.10.